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# AGGREGATE SPECIFICATIONS FOR STONE MASTIC ASPHALT (SMA)

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<b>16 Abstract</b> <p>Stone matrix or stone mastic asphalt (SMA) is one of the more promising mixes to come out of the European Study Tour. SMA mixes rely on a strong aggregate skeleton structure to support wheel loads and a high asphalt content to provide durability. Due to the stone-on-stone contact of properly designed SMA mixes, high quality, angular, rough-textured aggregates are typically recommended. Typical specification requirements are a maximum LA Abrasion of 30 percent, however, aggregates with and LA Abrasion as high as 40 percent have been successfully utilized.</p> <p>The objectives of the study were to evaluate Kansas aggregates for use in SMA mixes. Aggregates were selected that have LA Abrasions of less than 30 percent, 31 to 40 percent and more than 40 percent. SMA mixes were made from the aggregates and the extracted aggregate evaluated to determine percent degradation during laboratory compaction and testing. SMA samples were tested for performance using the Asphalt Pavement Analyzer to determine rutting performance. The results were analyzed to determine the relationships between aggregate degradation and LA Abrasion.</p> <p>Satisfactory SMA mixtures could not be made with the selected aggregates. All mixtures exceeded the maximum recommended flow value. The results indicated that the aggregates degraded extensively on the 9.5 mm sieve resulting in excess material accumulating on the 2.36 and/or 1.18 mm sieve. The excess material was generally in excess of the current Kansas Department of Transportation specification limit of 4 percent. SMA mixtures made with lower LA Abrasion aggregates showed less degradation. The finer the SMA gradation the less aggregate degradation occurred.</p>					
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## **Final Report**

By

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## **PREFACE**

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## ABSTRACT

Stone matrix or stone mastic asphalt (SMA) is one of the more promising mixes to come out of the European Study Tour. SMA mixes rely on a strong aggregate skeleton structure to support wheel loads and a high asphalt content to provide durability. Due to the stone-on-stone contact of properly designed SMA mixes, high quality, angular, rough-textured aggregates are typically recommended. Typical specification requirements are a maximum LA Abrasion of 30 percent, however, aggregates with an LA Abrasion as high as 40 percent have been successfully utilized.

The objectives of the study were to evaluate Kansas aggregates for use in SMA mixes. Aggregates were selected that have LA Abrasions of less than 30 percent, 31 to 40 percent and more than 40 percent. SMA mixes were made from the aggregates and the extracted aggregate evaluated to determine percent degradation during laboratory compaction and testing. SMA samples were tested for performance using the Asphalt Pavement Analyzer to determine rutting performance. The results were analyzed to determine the relationships between aggregate degradation and LA Abrasion.

Satisfactory SMA mixtures could not be made with the selected aggregates. All mixtures exceeded the maximum recommended flow value. The results indicated that the aggregates degraded extensively on the 9.5 mm sieve resulting in excess material accumulating on the 2.36 and/or 1.18 mm sieve. The excess material was generally in excess of the current Kansas Department of Transportation specification limit of 4 percent. SMA mixtures made with lower LA Abrasion aggregates showed less degradation. The finer the SMA gradation the less aggregate degradation occurred.

## TABLE OF CONTENTS

List of Tables	Page v
List of Figures	vii
Chapter 1. Plan of Study	1
General Problem Statement	1
Project Objectives	2
Chapter 2. Review of Literature	4
SMA and LA Abrasion	4
Sand Equivalent	4
SMA Literature Review	5
Chapter 3. Materials	16
Aggregates	16
Stabilizing Additive	18
Asphalt Cement	18
Chapter 4. SMA Mix Evaluation	19
Introduction	19
Mix Design Procedure	23
SMA Mix Designs	24
Chapter 5. Aggregate Degradation	41
Coarse SMA Gradation	41

	Page
Intermediate SMA Gradation	43
Fine SMA Gradation	47
LA Abrasion vs. Aggregate Degradation	57
Chapter 6. Performance and Moisture Sensitivity Testing	62
Chapter 7. Conclusions and Implementation.	65
References.	68

## LIST OF TABLES

	Page
Table 1. Completed SMA Projects.	9
Table 2. Stabilizing Additive & Volumetric Properties of SMA Projects.	9
Table 3. Coarse Aggregate Specifications.	11
Table 4. Fine Aggregate Specifications.	11
Table 5. Sources of Aggregates.	16
Table 6. Results of Physical Property Tests by KDOT.	17
Table 7. Gradation of Aggregates "As Received."	17
Table 8. SMA Gradations.	20
Table 9. Coarse SMA Gradation.	20
Table 10. Intermediate SMA Gradation.	22
Table 11. Fine SMA Gradation.	22
Table 12. Preliminary Mix Design Results, Franklin/Riley Co. Aggregates.	25
Table 13. Mix Design Results, Franklin/Riley Co. Aggregates.	26
Table 14. Preliminary Mix Design Results, Dolese Aggregates.	32
Table 15. Mix Design Results, Dolese Aggregates.	34
Table 16. Preliminary Mix Design Results, Formosa, KS Aggregates.	39
Table 17. Aggregate Degradation, Coarse SMA Gradation.	42
Table 18. Aggregate Degradation, Intermediate SMA Gradation, Franklin/Riley Co. Aggregates.	45
Table 19. Aggregate Degradation, Intermediate SMA Gradation, Dolese Aggregates.	46

	Page
Table 20. Aggregate Degradation, Fine SMA Gradation, Franklin/Riley Co.	
Aggregates.	49
Table 21. Aggregate Degradation, Fine SMA Gradation, Dolese Aggregates.	50
Table 22. Aggregate Degradation, Fine SMA Gradation, Formosa	
Aggregates.	51
Table 23. Mix Design Results, Franklin/Riley Co. Aggregates, no Chat.	53
Table 24. Aggregate Degradation, Fine SMA Gradation, Franklin/Riley Co.	
Aggregates, no Chat.	58
Table 25. Composition of Dense Graded HMA (SM-2C), Franklin/Riley Co.	
Aggregates.	63
Table 26. Mix Design Properties, Dense Graded HMA (SM-2C).	63
Table 27. APA Rut Test Results.	64

## LIST OF FIGURES

	Page
Figure 1. SMA Mineral Aggregate Gradations.	21
Figure 2. VTM vs. Asphalt Content, Franklin/Riley Co. Aggregates.	27
Figure 3. VMA vs. Asphalt Content, Franklin/Riley Co. Aggregates.	27
Figure 4. VFA vs. Asphalt Content, Franklin/Riley Co. Aggregates.	28
Figure 5. Unit Weight vs. Asphalt Content, Franklin/Riley Co. Aggregates.	28
Figure 6. Marshall Stability vs. Asphalt Content, Franklin/Riley Co. Aggregates.	29
Figure 7. Marshall Flow vs. Asphalt Content, Franklin/Riley Co. Aggregates.	29
Figure 8. VTM vs. Asphalt Content, Dolese Aggregates.	35
Figure 9. VMA vs. Asphalt Content, Dolese Aggregates.	35
Figure 10. VFA vs. Asphalt Content, Dolese Aggregates.	36
Figure 11. Unit Weight vs. Asphalt Content, Dolese Aggregates.	36
Figure 12. Marshall Stability vs. Asphalt Content, Dolese Aggregates.	37
Figure 13. Marshall Flow vs. Asphalt Content, Dolese Aggregates.	37
Figure 14. Aggregate Degradation, Coarse SMA Gradation.	44
Figure 15. Aggregate Degradation, Intermediate SMA Gradation.	48
Figure 16. Aggregate Degradation, Fine SMA Gradation.	52
Figure 17. VTM vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.	54
Figure 18. VMA vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.	54
Figure 19. VFA vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.	55
Figure 20. Unit Weight vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.	55

	Page
Figure 21. Marshall Stability vs. Asphalt Content, Franklin/Riley Co.	
Aggregates, no Chat.	56
Figure 22. Marshall Flow vs. Asphalt Content, Franklin/Riley Co. Aggregates,	
no Chat.	56
Figure 23. Effect of Chat on Aggregate Degradation, Franklin/Riley Co.	59
Aggregates.	
Figure 24. Aggregate Degradation by Mix.	60
Figure 25. Degradation vs. LA Abrasion.	61



# **CHAPTER 1**

## **PLAN OF STUDY**

### **GENERAL PROBLEM STATEMENT**

Stone matrix or stone mastic asphalt (SMA) is one of the more promising mixes to come out of the European Study Tour (1). SMA mixes rely on a strong aggregate skeleton structure to support wheel loads and a high asphalt content to provide durability. Fibers are utilized to hold the thicker films of asphalt, associated with SMA mixes, to the aggregate during production. SMA has been recommended for high traffic pavements as a solution to rutting and fatigue failures. Due to the stone on stone contact of properly designed SMA mixes, high quality, angular, rough textured aggregates are typically recommended.

The most common test for determining the suitability of an aggregate for use in SMA mixes is the LA Abrasion test. Typical specification requirements are a maximum LA Abrasion of 30 percent, however, aggregates with an LA Abrasion as high as 40 percent have been successfully utilized. Current Kansas Department of Transportation (2) specifications require the use of a minimum of 40 percent primary aggregates if the SMA will be used as a surface mix. The effect of using softer aggregates for the remainder of the aggregate has not been evaluated. At this time there are no definitive test methods for evaluation of aggregates for use in SMA. The SHRP gyratory compactor (SGC) and the U.S. Army Corps of Engineers gyratory testing machine (GTM) have both been utilized to evaluate aggregate performance. At this time, performance requirements have not been established for either

method. The use of SMA would have many positive effects on Kansas pavements if suitable aggregates could be found.

Other performance features than merit additional investigation include the moisture sensitivity or stripping potential of these mixes as well as the current sand equivalent requirement for SMA mixes.

## **PROJECT OBJECTIVES**

The objectives of the study are to evaluate Kansas aggregates for use in SMA mixes, evaluate moisture susceptibility and develop related SMA aggregate specification requirements. The objectives of this study will be met by completing the following tasks:

**Task 1. Review of Available Literature:** A review of the available literature will be conducted. This review will cover available previous work relevant to the scope of this study such as the work carried out by the National Center for Asphalt Technology (NCAT) and others on SMA. Emphasis will be placed on evaluating maximum LA Abrasion requirements and sand equivalent requirements.

**Task 2. Evaluate Aggregate Properties:** KDOT's test records will be evaluated to identify aggregates that have LA Abrasion loss of 0 to 30 percent, 31 to 40 percent and 41 to 50 percent. One aggregate from each LA Abrasion range would be selected for further testing and evaluation in combination with a primary aggregate. The primary aggregate and test aggregates would be selected in consultation with the project monitor.

**Task 3. SMA Mix Evaluation:** SMA mixes will be made from the aggregates selected in Task 2. Trial mixes will be made, the extracted aggregate gradation determined, and the percent degradation during laboratory compaction evaluated. SMA samples will be tested for performance using the Asphalt Pavement Analyzer to determine rutting performance. The performance data will be correlated to aggregate properties evaluated in Task 2. A dense graded mix using the same aggregates will be evaluated for comparison.

**Task 4. Evaluate Aggregate Resistance to Abrasion using Either Marshall or SGC Compaction:** Three SMA gradations will be selected for use based on current KDOT specifications (2). The aggregates selected in Task 2 will be evaluated for percent degradation by compacting using Marshall compaction. The results will be analyzed and the relationships between percent loss in LA Abrasion, SGC or Marshall compaction determined.

**Task 5. SMA Moisture Sensitivity Evaluation:** The original plan called for SMA to be made from the aggregates evaluated in Task 2. The mixes would be evaluated for moisture sensitivity using the Asphalt Pavement Analyzer. After consultation with the project monitor, it was decided to defer the majority of the testing originally scheduled for this task to K-TRAN KU:99-3, *Evaluation of Anti Stripping Agents using the Asphalt Pavement Analyzer* due to the poor performance of the aggregates selected in Task 2.

**Task 6. Implementation Statement:** A final report based on the findings from Tasks 1-4 will be prepared. Conclusions regarding the suitability of Kansas aggregates for use in SMA and trial SMA aggregate specifications will be developed and implementation plans proposed.

## **CHAPTER 2**

### **REVIEW OF LITERATURE**

(Task 1)

#### **SMA and LA ABRASION**

A comprehensive review of the literature on SMA was prepared by NCAT as a part of NCHRP 9-8 and can be found in the report *NCHRP 9-8 Designing Stone Matrix Asphalt Mixtures, Volume 1 - Literature Review* (3). Therefore, a comprehensive literature was not repeated. None of the papers in the review by NCAT specifically discussed the evaluation of LA Abrasion requirements for SMA other than the suggested maximums.

#### **SAND EQUIVALENT**

A second objective of the literature was to review work on the sand equivalent test. A review of the literature for the effects of sand equivalent of mix performance was not productive. Little literature of any consequence was found. One exception to this was a paper by Aschenbrener and Zamora (4). The authors reviewed specialized tests for aggregates in asphalt pavements. The authors concluded that there was a poor correlation between sand equivalent value and field performance with respect to stripping. This is not surprising because the sand equivalent test measures the amount of clay size particles, not the amount of clay minerals (clay) in a sample. Stripping has been associated with plastic clay fines. Clay minerals are plastic, clay size particles not composed of clay minerals are not plastic. It is the authors opinion that the sand equivalent test was a useful test when many

mixes were made with pit run aggregates and the majority of clay size particles would be composed of clay minerals. Dust created from crushing aggregates results in clay size particles, not necessarily clay minerals. The sand equivalent test cannot distinguish between the two and therefore is not a good indicator of performance. As pointed out by the author and others (4), high amounts of clay size particles are not desirable either. Aschenbrenner (4) reported high dust coatings on aggregates as a major cause of poor performance.

## **SMA LITERATURE REVIEW**

The following is a literature review conducted by Kenneth Kekessie on SMA as a part of his Master's thesis (5).

Stone matrix or stone mastic asphalt (SMA) gained popularity in the United States after the European Study Tour of 1990. The tour was arranged to exchange ideas and experience with highway agencies and the construction industry in Europe on design methods as well as production and placement of asphalt pavements. The group consisted of officials from American Association of State Highway and Transportation Officials (AASHTO), Federal Highway Administration (FHWA), National Asphalt Pavement Association (NAPA), Strategic Highway Research Program (SHRP), Asphalt Institute (AI) and the Transportation Research Board (TRB).

The group visited six European nations namely, Sweden, Denmark, Germany, France, United Kingdom and Italy because of similarities they share with the United States. They are all industrialized nations, have extensive highway and road systems and motor vehicles are

increasingly relied upon for movement of people and goods. All the nations visited have modern, capable highway agencies and a mature construction industry.

German road contractors first used SMA, the English translation of "split mastix asphalt" (6), in Germany in the 1960's. Its use is now prevalent in many European countries. The development of SMA was necessitated by the need for a high performance wearing surface that was capable of resisting rutting and abrasion under heavy traffic loads. SMA is composed of crushed stone aggregates, asphalt cement and a stabilizing additive, normally cellulose fiber or mineral fiber.

Since it was first introduced in Europe, SMA has provided a rut resistant pavement surface that has resulted in about a 25-30% increase in the service life of such pavements (7). SMA differs from the traditional dense graded aggregate mixes in that it is a gap graded mixture which contains a large amount of coarse aggregate, i.e. aggregates with a minimum particle size of 4.75mm. The gap aggregate gradation is the reason for the rut resisting ability of SMA mixes because it provides stone-on-stone contact which forms a stone skeleton after compaction that is capable of resisting further densification under traffic loads and thus provide resistance against rutting.

Wolfgang et al. (6) listed some of the advantages of properly designed and produced SMA pavements as follows:

- 1) The stone skeleton gives the mix excellent shear resistance due to its high internal friction.
- 2) The voidless mastic, which is rich in binder, provides significant durability and adequate resistance to cracking.

3) The increased amount of large sized aggregates provides superior resistance to the wear of studded tires.

4) Good skid resistance and proper light reflection are enhanced in SMA mixes because of the rough surface texture of such mixes.

SMA mixes have been used in the United States (US) since the early part of this decade. Traffic rates on pavements with such mixes have been high and this has resulted in large amounts of traffic loadings on SMA pavements in a short period of time.

The FHWA, in association with various state Departments of Transportation, started a series of SMA trial pavements in five states in 1991 (7). The states that were used for the initial study were Michigan, Wisconsin, Georgia, Missouri and Indiana. These trials were to serve as a basis for evaluating the feasibility of constructing SMA pavements in the US and also to evaluate the performance of SMA mixes as compared to the traditional dense graded aggregate mixes. Bukowski (7) listed some of the findings of the initial evaluation conducted in 1991 as follows:

1) The 4.75mm sieve controls the existence of appropriate stone-on-stone contact.

The percentage of the coarse aggregate passing this sieve should not exceed 30%.

2) In order to maximize stone-on-stone contact, the amount of flat and elongated aggregates should be controlled by limiting the amount of coarse aggregates with a length to width ratio of 3 to 1 to about 20% of the total aggregate.

3) The mineral dust has a significant effect on the behavior of SMA mixes and thus the portion of the mineral dust less than 0.020 mm in size should be limited to about 3% of the total aggregate.

Table 1 shows the SMA projects completed in the US by 1991 and the type of materials used in construction, the thickness of the pavement and the gradation of the aggregates. Table 2 shows the stabilizing additive used, the air void content and the voids in mineral aggregate of the mixes.

Brown et al. (8) stated that initial SMA design in the U.S. attempted to duplicate the techniques employed by European designers. However, because of differences in material properties and construction practices it proved to be unrealistic. The majority of SMA mixed currently in the U.S. were made using moderately stiff to stiff asphalts usually having a penetration in the 60 to 80 range (8). Temperature variations in this country require that the use of other grades of asphalt may be more appropriate for the various climatic regions in the US.

### **Mixture Design**

Wolfgang et al. (6) stated that the three principal conditions that must be satisfied during the design of SMA mixes are:

- 1) The coarse aggregates must be able to form a stone skeleton with firm contact between the aggregates.
- 2) The coarse particles should be held together by a voidless mastic such as asphalt cement.
- 3) The mastic should be stable enough to prevent drain down from the coarse particles during storage, haulage and placement of the mix.

Table 1. Completed SMA Projects (3).

State	Michigan	Wisconsin	Georgia	Missouri	Indiana
Section	1.5" Surface	1.5" Binder	1.5" Surface 1.5" Binder	1.5" Surface	1.5" Surface
Gradation	Percent Passing				
19.0 mm	100	100	100	100	100
12.5 mm	94	92	100 64	96	95
9.5 mm	73	72	78 39	76	31
4.75 mm	35	28	36	34	25
2.36 mm	24	21	23 22	20	24
1.18 mm	19	17		15	15
0.600 mm	16	15		13	
0.300 mm	14	14	15 18	12	
0.150 mm	12	12		11	
0.075 mm	10	11	10 10	10	10

Table 2. Stabilizing Additive &amp; Volumetric Properties of SMA Projects (3).

State	Michigan	Wisconsin	Georgia	Missouri	Indiana
Additive	Cellulose Fiber, Pell, & Polyolefin	Polyolefin	Mineral fiber/ Modifier	Cellulose & Mineral fibers	Multigrade 20-30
VTM	3%	3%	3.5%	4%	3%
VMA	16%	17%	18% 17.5%	18%	17%
Plant	Drum & Batch	Batch	Batch	Batch	Batch
Quantity	3,000 tons	1,000 tons	3,000 tons	1,000 tons	1,000 tons

The first condition implies that there must exist enough void space in the compacted mixture to accommodate the mastic and the required air voids in the compacted mix. Furthermore, the density of the coarse aggregate in the compacted mix should be nearly the same as the coarse aggregate compacted separately.

The voidless nature of the mastic suggested by the second condition is that the durability of the mastic is dependent on the degree of compaction and since the volume of mastic is less than the volume of aggregate it is difficult to compact the mastic sufficiently.

The mastic is principally composed of asphalt cement, which is a viscoelastic material with low viscosity at the mixing and compaction temperatures. The mastic will therefore drain off the coarse aggregates after it has been mixed. In Europe adding fibers to the mix has successfully stabilized the mastic (6). The type of stabilizer used has been of concern to both engineers and the general public. Initial SMA mixes developed in Germany were stabilized with asbestos fiber but strong public opinion against its use led to the search for alternative types of stabilizers which provided the mix with the same or better qualities than the asbestos fiber. In Sweden a cellulose fiber has been developed by a company known as NCC, and it is marketed under the trademark name "Viacotop" (9).

### **Material Specifications**

NCAT's (8) material specifications are the ones currently in use by most state DOT's. The specifications for coarse and fine aggregates are shown in Tables 3 and 4 respectively.

Table 3. Coarse Aggregate Specifications (8).

Test	Method	Specification Maximum	Specification Minimum
LA Abrasion	AASHTO T96	30	-
Flat & Elongated	ASTM D4791 Section 8.4		
% 3 to 1		20	-
% 5 to 1		5	-
Absorption, %	AASHTO T85	2	-
Soundness (5 cycles)	AASHTO T85		
Sodium Sulfate		15%	-
Magnesium Sulfate		20%	-
Crushed Content	N/A		
One face		-	100%
Two faces		-	90%

Table 4. Fine Aggregate Specifications (8).

Test	Method	Specification Maximum	Specification Minimum
Soundness, % Loss	AASHTO T104		
Sodium Sulfate		15	-
Magnesium Sulfate		20	-
Angularity, %	AASHTO TP33	-	45
Liquid Limit, %	AASHTO T90	25	-
Plasticity Index	AASHTO T90	Non-plastic	

***Mineral Filler***

Mineral filler used should consist of finely divided mineral matter such as rock or limestone dust, which must be sufficiently dry to flow freely and not contain any organic impurities. It must also have a Plasticity Index (PI) of not greater than 4 and should meet the requirements of AASHTO M17, (8).

***Asphalt Cement***

Asphalt cement used should meet the requirements of AASHTO M 226, Table 2 or AASHTO MP1. In most areas, it may be prudent to use one grade stiffer than is normally employed (8).

***Stabilizing Additive***

The stabilizer used may be cellulose fiber, mineral fiber, or polymer. It is added to the mixture to prevent the draining off of the asphalt cement from the coarse aggregate surfaces during mixing and compaction. Dosage rate for cellulose fiber is 0.3% by total mixture weight. For mineral fiber the dosage rate is 0.4% by total mixture weight. The amount of polymer added is the amount suggested by the manufacturer or determined from past experience. An allowable tolerance of fiber dosage is about +/- 10% of the required fiber weight (8).

### *Determination of Stone-on-Stone Contact*

Brown et al. (8) stated that satisfactory performance of SMA depends on adequate stone-on-stone contact. To determine the existence of stone-on-stone contact, the voids in the coarse aggregate fraction (+4.75mm) are determined using the dry-rodded technique in accordance with AASHTO T19. The dry rodded unit weight ( $\gamma_s$ ) of the coarse aggregate is then substituted in the formula shown below to determine the voids in the coarse aggregate ( $VCA_{dry}$ ) in the dry rodded condition.

$$VCA_{dry} = ((Gsb_{coarse} * \gamma_w - \gamma_s) / (Gsb_{coarse} * \gamma_w)) * 100$$

Where:

$\gamma_s$  = Unit weight of the coarse aggregate fraction in the dry rodded condition  
(kg/m<sup>3</sup>)

$\gamma_w$  = Unit weight of water (999 kg/m<sup>3</sup>)

$Gsb_{coarse}$  = bulk specific gravity of the coarse aggregate

The voids in the coarse aggregate of the compacted mix ( $VCA_{mix}$ ) is determined from the bulk specific gravities of the mix ( $Gmb$ ) and coarse aggregate ( $Gsb_{coarse}$ ).

$$VCA_{mix} = 100 - (Gmb/Gsb_{coarse}) * Pca$$

Where:

$Pca$  = Percentage of coarse aggregate in the mix.

Stone-on-stone contact exists when the VCA of the mix ( $VCA_{mix}$ ) is less than or equal to the VCA of the coarse aggregate fraction ( $VCA_{dry}$ ).

***Optimum Asphalt Content***

The optimum asphalt content of the compacted mixture is determined using the NAPA (10) method which is outlined below:

- 1) Determine the AC content required to produce 4% voids total mix (VTM) in the mixture from a plot of VTM versus Asphalt Content.
- 2) Determine the following properties at this optimum asphalt content by referring to plots of Marshall Stability, Flow and Voids in Mineral Aggregate (VMA) versus asphalt content.
- 3) Compare each of these values against the specification values and if all are within the specification, then the preceding asphalt content is satisfactory. If any of these properties is outside the specification range, the mixture is redesigned.
- 4) If all the specification criteria are met, the AC content determined at 4% VTM is the optimum asphalt content.

**SMA Performance**

SMA has only recently been used in the US; therefore, there is limited data its performance. Consequently, states that have SMA pavements are being encouraged by FHWA to monitor performance and collect data for evaluation of the existing pavements. In some states SMA test sections have been placed adjacent to Strategic Highway Research Project (SHRP) special pavement sections and they will be monitored and evaluated as part of the long-term performance program for the next 15 years (7). Research is still being performed in the laboratory on the resistance of SMA to pavement deformation using the French Rutting

Tester, Hamburg Wheel Tracking Device and the Georgia Loaded Wheel Tester. A number of organizations are also hoping to be able to predict SMA performance by analyzing indirect tensile strength and resilient modulus (7).

Quality control and evaluation of bituminous pavement layers often have to be performed by highway engineers. In many cases, determination of the mix composition is insufficient for an evaluation of the mix properties. However, mechanical properties such as fatigue strength and stiffness modulus can provide much applicable information.

The mechanical properties of bituminous mixes can be determined by various methods, but these require special equipment and specimens of a particular configuration. The stiffness modulus and fatigue properties of different types of bituminous pavement layers have been determined in the laboratory. Indirect tensile tests (ITT) have been conducted primarily because it is a relatively simple and rapid test to conduct. However, initial results (8) suggest that these tests may not be very predictive of SMA performance potential. Static and dynamic creep determinations may offer more promise (7).

Entering the seventh year of use in the US, interest in SMA is still very high. It is expected that with development of SHRP, refinement of SMA will continue to progress and to provide better understanding of Stone Matrix Asphalt, the European pavement innovation.

## CHAPTER 3

### MATERIALS

#### (Task 2)

#### AGGREGATES

Four coarse aggregates, two fine aggregates and one mineral filler were selected by the project monitor and Principal Investigator (PI) to determine their suitability for use in SMA mixtures. The aggregates were selected to represent the range of limestone aggregates found in Kansas. The sources of the aggregates are shown in Table 5.

Table 5. Sources of Aggregates.

Gradation	Parent Material	Source
CS-1	Limestone	Martin-Marietta, Riley Co.
CS-1	Limestone	Dolese (K-254 SMA Mix)
CS-1	Chalk	Formoso, KS
CS-1P	Chat	KDOT
ManSand	Limestone	Fogle Quarry, Franklin Co.
CS-2	Limestone	Dolese (K-254 SMA Mix)
MFS-2	Limestone	Dolese (K-254 SMA Mix)

The physical properties of the aggregates were either supplied by KDOT or determined in the University of Kansas Bituminous Laboratory. The results of the physical property tests are shown in Table 6. The gradation of the aggregates, as received, are shown in Table 7. The Dolese materials (CS-1 & CS-2) were supplied by KDOT broken down on individual sieve sizes; therefore, as received gradations could not be determined. The

Table 6. Results of Physical Property Tests by KDOT.

Material	L A Abrasion	Specific Gravity AASHTO T84 or T85		
	AASHTO T96	Bulk	Apparent	Absorption
CS-1P, Chat	N/T	2.54	2.63	1.2%
CS-1, Dolese	22	2.67	2.71	0.7%
CS-2, Dolese	N/A	2.55	2.76	3.3%
MFS-2, Dolese	N/A	N/A	2.77	N/A
CS-1, Riley Co.	35	2.51	2.66	2.2%
ManSand, Franklin Co.	N/A	2.58	2.71	1.9%
CS-1, Formoso, KS	46	1.82*	2.69*	17.7%*

\* Test Performed by KU.      N/A = Not Applicable.      N/T = Not Tested.

Table 7. Gradation of Aggregates "As Received."

Sieve Size (mm)	Chat CS-1	Dolese Aggregates			Riley Co.	Franklin Co.	Formoso, KS
		CS-1	CS-2	MFS-2	CS-1	ManSand	CS-1
		Percent Retained					
25.0	0			0	0	0	0
19.0	0			0	0	0	15
12.5	0			0	20	0	43
9.5	0			0	50	0	55
4.75	85	N/A	N/A	0	92	0	73
2.36	100			0	95	18	81
1.18	100			0	95	57	85
0.600	100			0	96	76	89
0.300	100			0	96	89	91
0.150	100			22	96	94	92
0.075	100			39	96	96	93

N/A = Not Available, Materials Supplied Sieved.

remaining aggregates were broken down on individual sieve sizes down through the 0.300 mm sieve and recombined to make the appropriate SMA mixtures.

#### **STABILIZING ADDITIVE**

The stabilizing additive utilized was a natural cellulose fiber supplied by KDOT. The cellulose fiber was incorporated into the mix at a dosage rate of 0.3% by weight of the total mix.

#### **ASPHALT CEMENT**

The asphalt cement used was an AC-20, supplied by Total. The AC-20 is one grade stiffer than what is typically used in Kansas. This followed the recommendations of NCAT (8).

The specific gravity of the asphalt cement was 1.030.

**CHAPTER 4**  
**SMA MIX EVALUATION**  
**(Task 3)**

**INTRODUCTION**

Three different aggregate blends were selected for evaluation within the current KDOT SMA specification (2). The blends were a coarse gradation, an intermediate gradation and a fine gradation. The gradations along with the KDOT specification are shown in Table 8 and graphically in Figure 1.

Mixtures were made to each gradation using the three different coarse aggregates. Each mixture contained 40% primary aggregate (chat) as required for a surface or wearing course mixture. The objective of this study was to evaluate the suitability of Kansas aggregates for SMA. SMA develops its strength from a strong aggregate skeleton and the aggregate properties are important to the successful performance of SMA. Therefore, the mineral filler for each mixture was from a single source to better evaluate the effects of the aggregate skeleton on SMA performance. The “as received” gradations of the aggregates are shown in Table 7. The blends of the individual aggregates for the coarse, intermediate and fine gradations are shown in Tables 9-11.

Due to the suspected soft nature of Kansas limestone aggregates mixtures were made using Marshall compaction rather than SGC compaction. Brown (12 ) reported that SGC compaction resulted in more aggregate breakdown than Marshall compaction and recommended Marshall compaction for agencies with marginal aggregates.

Table 8. SMA Gradations.

Sieve Size (mm)	KDOT SMA Spec.			Fine
	Coarse	Intermediate		
	(Percent Retained)			
19.0	0	0	0	0
12.5	13.5	13.5	7.0	5-15
9.5	45.0	39.8	29.8	25-60
4.75	82.0	76.4	69.8	67-85
2.36	86.0	80.4	75.0	71-89
1.18	88.0	81.7	77.1	
0.600	89.0	82.3	81.4	81-91
0.300	90.0	83.0	82.2	82-92
0.150	92.2	86.7	86.7	
0.075	93.9	89.6	89.6	88-94

Table 9. Coarse SMA Gradation.

Material Pct. In Blend	Chat 40	CS-1 30	CS-2 20	Filler 10	Combined Gradation
Sieve Size (mm)	(Percent Retained)				
19.0	0	0	0	0	0
12.5	0	45	0	0	13.5
9.5	0	100	75	0	45.0
4.75	90	100	80	0	82.0
2.36	100	100	80	0	86.0
1.18	100	100	90	0	88.0
0.600	100	100	95	0	89.0
0.300	100	100	100	0	90.0
0.150	100	100	100	22	92.2
0.075	100	100	100	39	93.9

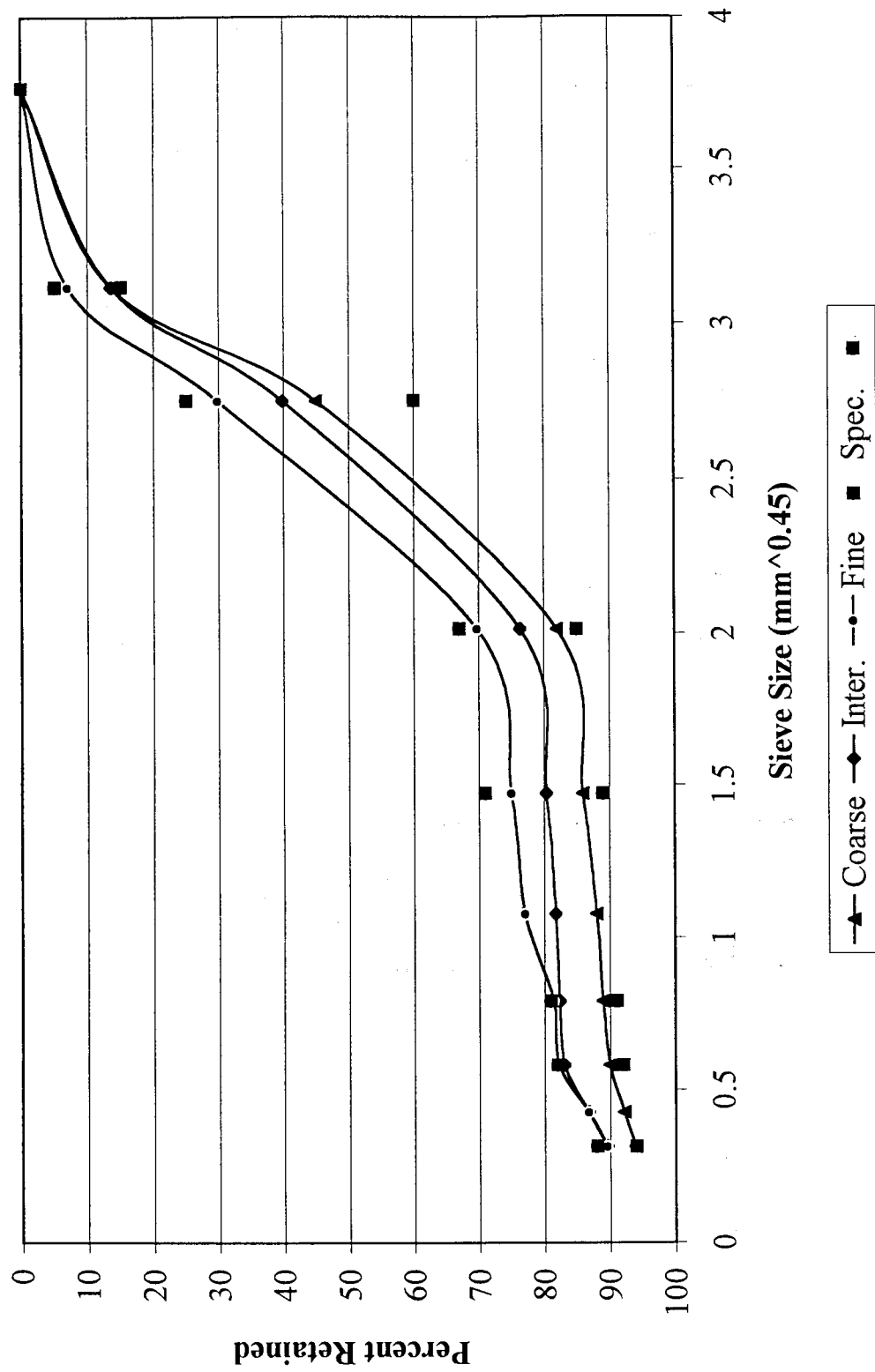


Figure 1. SMA Mineral Aggregate Gradations.

Table 10. Intermediate SMA Gradation.

Material Pct. In Blend	Chat 40	CS-1 30	CS-2 13	Filler 17	Combined Gradation
Sieve Size (mm)	(Percent Retained)				
19.0	0	0	0	0	0
12.5	0	45	0	0	13.5
9.5	0	100	75	0	39.8
4.75	90	100	80	0	76.4
2.36	100	100	80	0	80.4
1.18	100	100	90	0	81.7
0.600	100	100	95	0	82.4
0.300	100	100	100	0	83.0
0.150	100	100	100	22	86.7
0.075	100	100	100	39	89.6

Table 11. Fine SMA Gradation.

Material Pct. In Blend	Chat 40	CS-1 35	CS-2 8	Filler 17	Combined Gradation
Sieve Size (mm)	(Percent Retained)				
19.0	0	0	0	0	0
12.5	0	20	0	0	7.0
9.5	0	85	0	0	29.8
4.75	87	100	0	0	69.8
2.36	100	100	0	0	75.0
1.18	100	100	26	0	77.1
0.600	100	100	80	0	81.4
0.300	100	100	90	0	82.2
0.150	100	100	100	22	86.7
0.075	100	100	100	39	89.6

## **MIX DESIGN PROCEDURE**

The mix design procedure followed was the method recommended by NCAT (8). In order to save time and materials, a preliminary mix design was made using the Dolese and Franklin/Riley Co. aggregates. The preliminary mix design consisted of compacting two samples at one asphalt content (6.0 - 7.0%) from each aggregate source to the coarse, intermediate and fine SMA gradations. The preliminary results were analyzed and if it appeared possible to make a SMA mix meeting specifications, additional asphalt contents were selected and additional samples compacted.

Samples were batched (1100g) to the appropriate gradation and the aggregates heated overnight at 160°C. The asphalt cement was heated to 145°C and samples mixed and compacted at 150°C. Samples were compacted to 50 blows per side using an automatic Marshall hammer with a slanting compaction foot and rotating base. After compaction the samples were allowed to cool and then tested for bulk specific gravity (ASTM D2726) and Marshall stability and flow (ASTM D1559). After Marshall stability and flow testing the samples were heated until they could be easily broken apart with a spatula and then tested for asphalt content by ignition in accordance with ASTM PS90. After ignition testing the samples were tested for gradation analysis using ASTM C117 & C136. Samples of loose mix were tested for maximum specific gravity in accordance with ASTM D2041.

After analysis of the preliminary mix design data, additional samples were made at other asphalt contents if warranted and the testing sequence repeated. The results were compared to the NCAT (8) recommended SMA specifications at the optimum asphalt content which corresponds to 3.5 - 4.0% VTM.

## **SMA MIX DESIGNS**

### **Franklin/Riley Co. Aggregates**

#### ***Preliminary Mix Design***

The results of the preliminary mix designs for the Franklin/Riley Co. aggregates for the coarse, intermediate and fine gradations are shown in Table 12. The preliminary mix design samples for the coarse SMA gradation, compacted at 7.0% asphalt, had high VTM, high voids in the mineral aggregate (VMA) and low voids filled with asphalt (VFA), which indicates more asphalt is needed in the mixture. The Marshall stabilities were very low, 3835 N, and the flows were high, 21, indicating too much asphalt or excessive fine aggregate. Significant aggregate breakdown was evident by observing the samples after Marshall stability testing.

Based on the conflicting test results and observations of coarse aggregate breakage, excessive aggregate breakdown was suspected. The high VMA indicated a need for a gradation closer to the maximum density line and/or more mineral filler. The intermediate gradation is the coarse gradation with more mineral filler and the fine gradation is closer to the maximum density line. Because the mix properties of the coarse gradation were so far out of specification and the indicated changes in gradations were similar to the intermediate and fine gradations, no additional samples were made to the coarse gradation for the Franklin/Riley Co. aggregates.

The preliminary results for the intermediate and fine gradations showed promising results with VTMs near acceptable range and VMAs near or above the minimum

recommendation of 17%. Therefore, additional samples were made for the intermediate and fine gradations and complete mix designs performed.

Table 12. Preliminary Mix Design Results, Franklin/Riley Co. Aggregates

Gradation	Asphalt Content (%)	Bulk Specific Gravity	Maximum Specific Gravity	Unit Weight (kN/m <sup>3</sup> )	VTM (%)	VMA (%)	VFA (%)	Marshall	
								Stability (N)	Flow (0.1 mm)
Coarse	7.0	2.187	2.420	21.45	9.6	20.6	53.2	3835	21
Intermediate	6.5	2.237	2.363	21.94	5.3	18.8	71.5	8175	23
Fine	6.5	2.297	2.389	22.53	3.9	16.4	76.6	4710	25

### *Intermediate Gradation*

The full mix design results for the intermediate gradation are shown in Table 13 and graphically in Figures 2-7. As shown in Figure 6, Marshall stabilities were above the minimum recommended value of 6200 N at all asphalt contents. The flows (Figure 7) were well above the recommended maximum of 16. However, NCAT (8) recommends that SMA mixtures not be rejected due to high flow alone. The VFAs were within reasonable limits (Figure 4) and the VMAs (Figure 3) were above the recommended minimum of 17. The VTMs (Figure 2) indicate an optimum asphalt content of approximately 7.5%. The plot of VMA versus asphalt content (Figure 3) shows that optimum asphalt content lies on the wet side of the VMA curve. The Asphalt Institute (13) recommends that mixtures not be

Table 13. Mix Design Results, Franklin/Riley Co. Aggregates

Asphalt Content (%)	Bulk Specific Gravity	Maximum Specific Gravity	Unit Weight (kN/m^3)	VTM (%)	VMA (%)	VFA (%)	Marshall	
							Stability (N)	Flow (0.1 mm)
Intermediate Gradation								
6.0	2.234	2.379	21.92	6.1	18.4	67.0	11285	26
6.5	2.237	2.363	21.94	5.3	18.8	71.5	8175	23
7.0	2.240	2.347	21.97	4.6	19.1	76.0	9195	29
Fine Gradation								
6.0	2.299	2.406	22.56	4.4	15.9	72.1	5115	27
6.5	2.297	2.389	22.53	3.9	16.4	76.6	4705	25
7.0	2.298	2.372	22.55	3.1	16.8	81.5	4335	27

designed on the wet side of the VMA curve. On the wet side of the VMA curve the asphalt cement is preventing good aggregate contact and the mixture is approaching a plastic condition. The reduction of Marshall stability with asphalt content (Figure 6) indicates the mix is on the plastic side as well. Marshall stability samples showed signs of aggregate breakdown as well. Although the mixture met the minimum requirements for an SMA mixture, the mixture would not be recommended by the PI because the optimum asphalt content is on the wet side of the VMA curve.

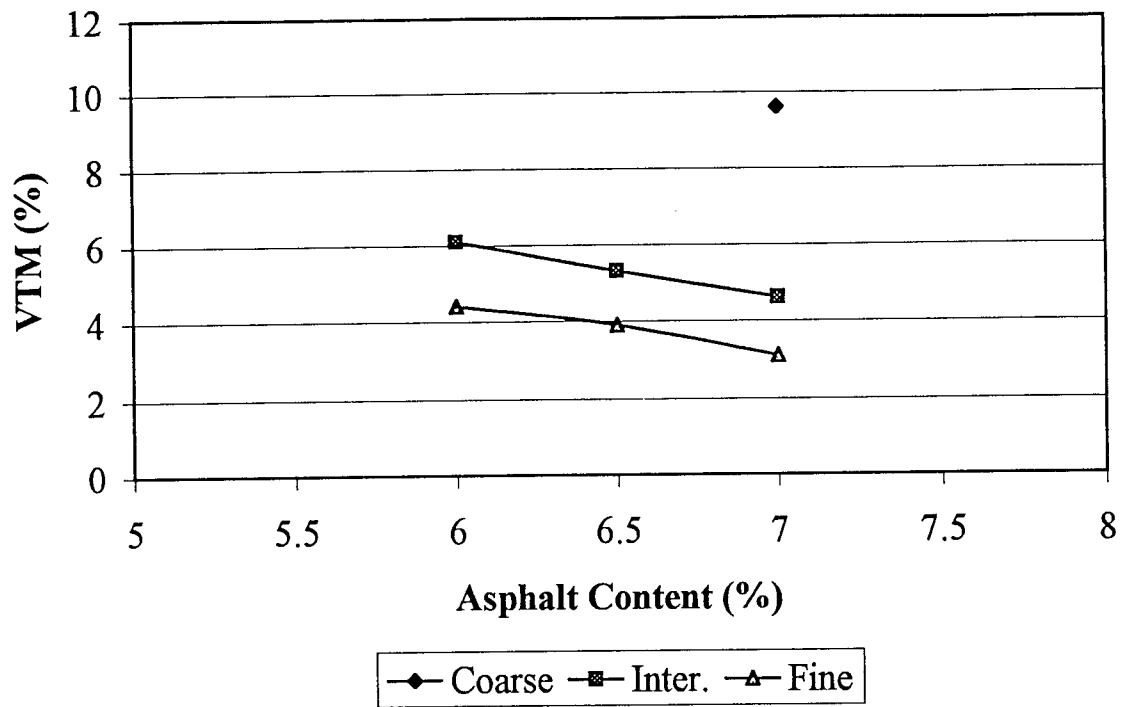


Figure 2. VTM vs. Asphalt Content, Franklin/Riley Co. Aggregates.

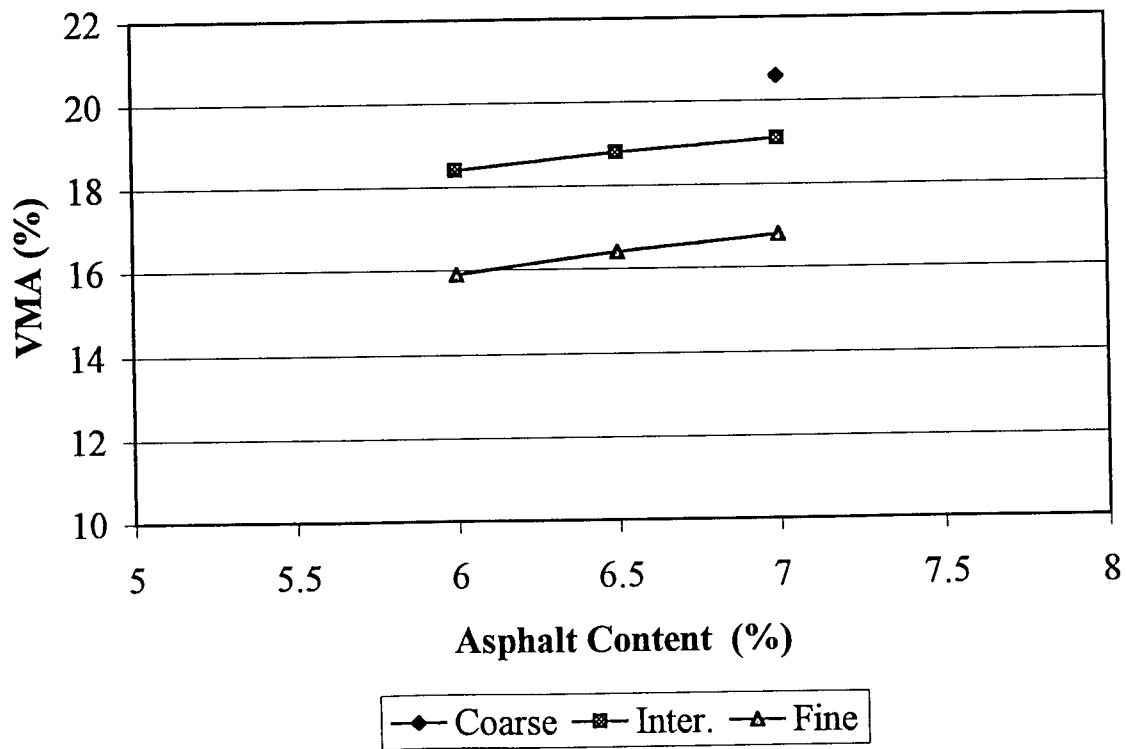


Figure 3. VMA vs. Asphalt Content, Franklin/Riley Co. Aggregates.

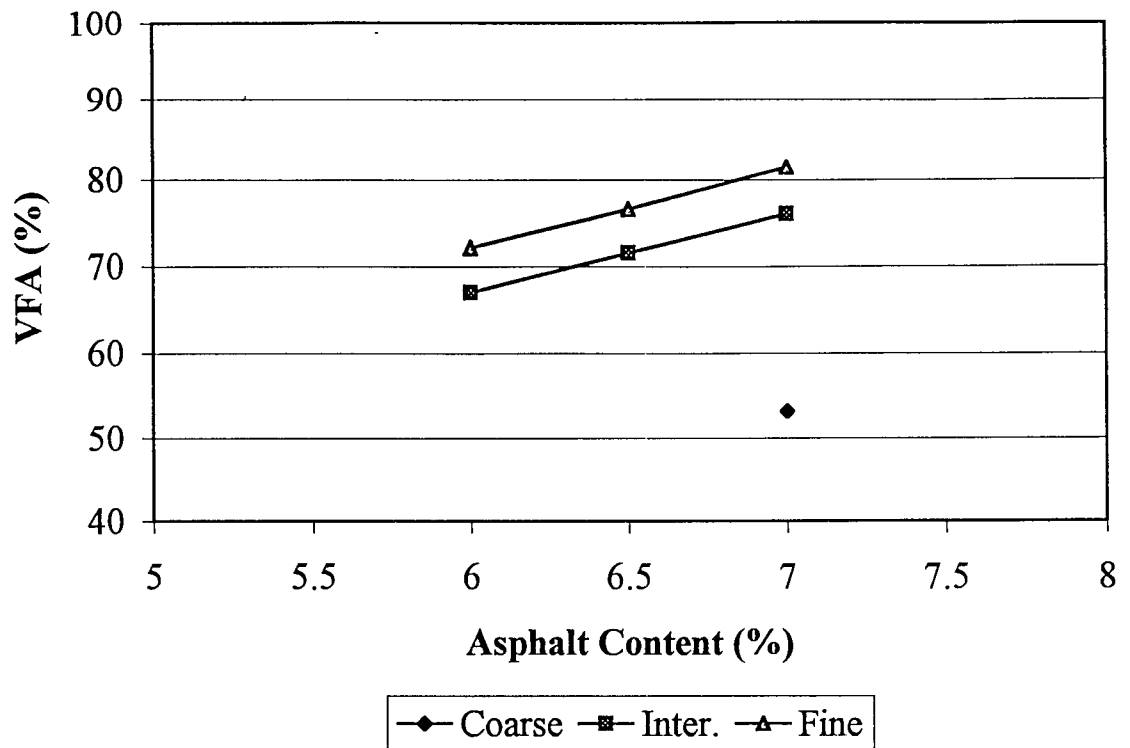


Figure 4. VFA vs. Asphalt Content, Franklin/Riley Co. Aggregates.

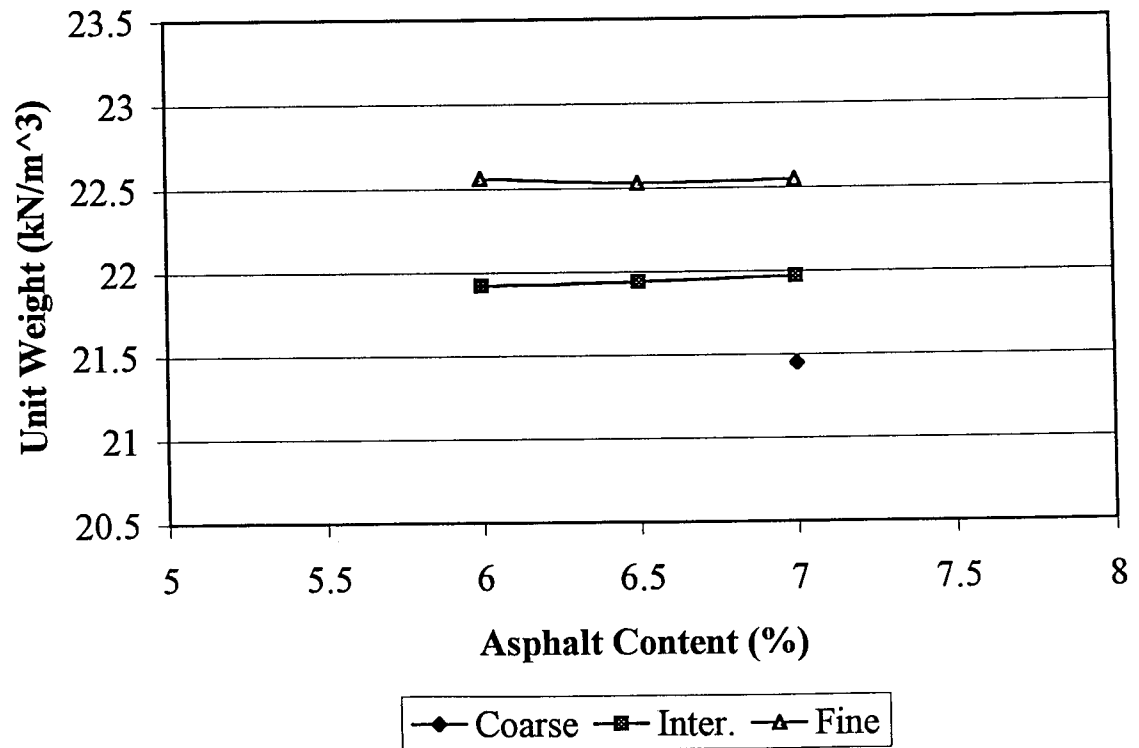


Figure 5. Unit Weight vs. Asphalt Content, Franklin/Riley Co. Aggregates.

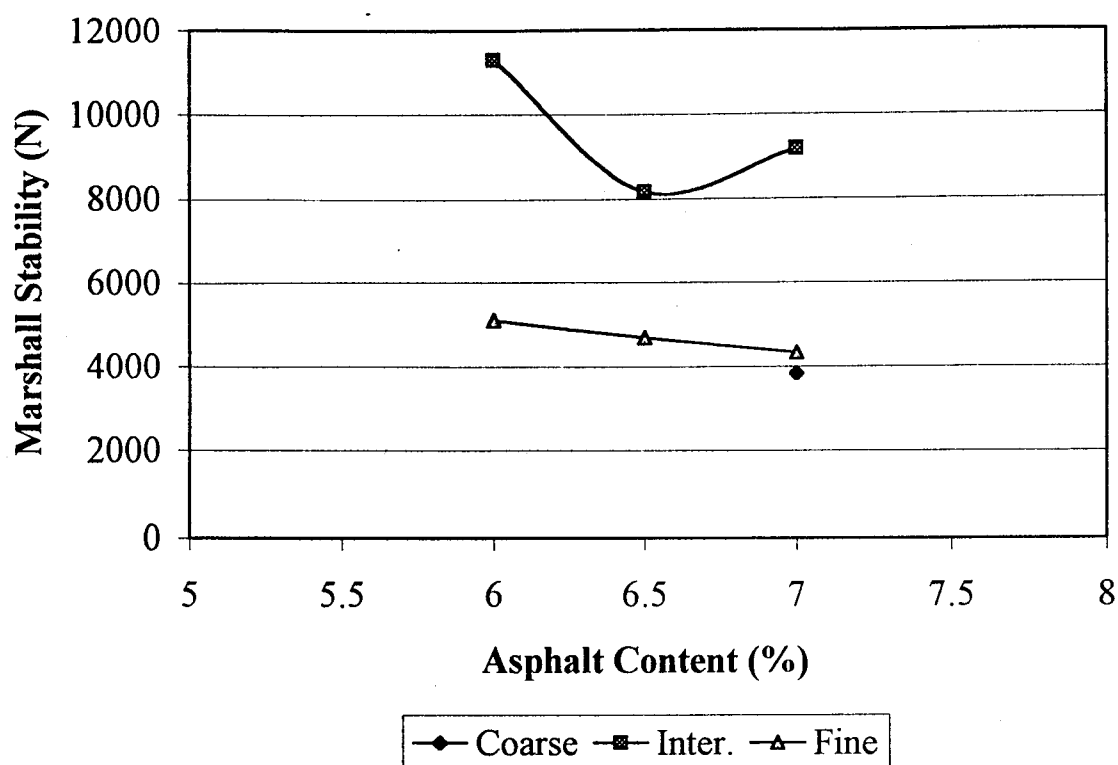


Figure 6. Marshall Stability vs. Asphalt Content, Franklin/Riley Co. Aggregates.

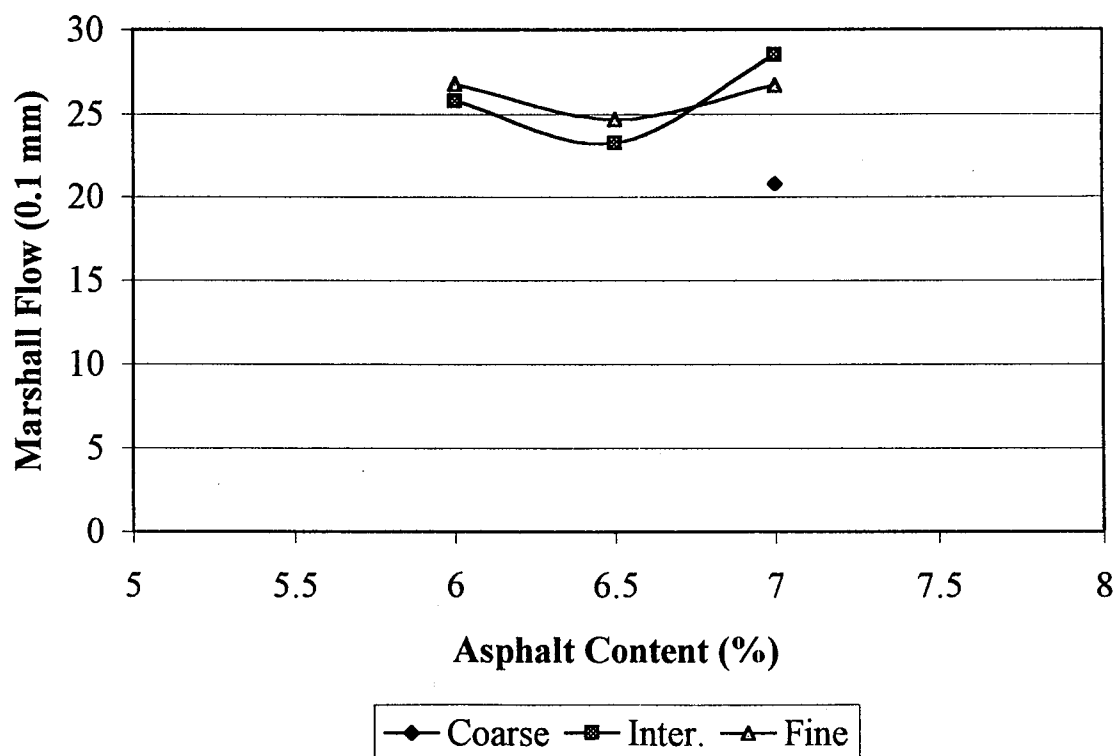


Figure 7. Marshall Flow vs. Asphalt Content, Franklin/Riley Co. Aggregates.

### ***Fine Gradation***

The results of the mix design for the fine gradation with the Franklin/Riley Co. aggregates are shown in Table 13 and graphically in Figures 2-7, as well. As shown in Figure 6, none of the samples met the minimum Marshall stability at any asphalt content and the flows were above the recommended maximum of 16 (Figure 7). Figure 3 shows the VMAs were below the minimum recommended value of 17%. The VFAs (Figure 4) were acceptable. Optimum asphalt content corresponding to 4.0% VTM (Figure 2) was 6.5%. As with the intermediate mix, the optimum asphalt content was on the wet side of the VMA curve (Figure 3) and the decreasing Marshall stability with increasing asphalt content (Figure 6) indicates the mix is plastic. Aggregate breakdown was visible in the Marshall stability samples.

### ***Stone-on-Stone Contact***

The fine gradation was tested for stone-on-stone contact. This is evaluated by comparing the volume of the coarse aggregate in the mixture to the volume of the coarse aggregate fraction determined from the dry rodded unit weight test (AASHTO T19) (8). If the voids in the coarse aggregate of the mix ( $VCA_{mix}$ ) is less than the VCA of the coarse aggregate fraction ( $VCA_{dry}$ ), stone-on-stone contact exists. The  $VCA_{mix}$  is determined from the following formula:

$$VCA_{mix} = 100 - (Gmb/Gsb_{coarse}) * Pca$$

where:  $Gmb$  = bulk specific gravity of compacted mixture,

$Gsb_{coarse}$  = bulk specific gravity of the coarse aggregate

$Pca$  = percentage of coarse aggregate in mix.

The  $VCA_{mix}$  was less than the  $VCA_{dry}$  of the coarse aggregate fraction so stone-on-stone contact existed. The intermediate and coarse gradations would have stone-on-stone contact as well as they contained more coarse aggregate.

### ***Summary***

None of the Franklin/Riley Co. mixtures met all the requirements for an SMA mixture. The intermediate mixture came closest, but was not deemed a satisfactory mixture because the optimum asphalt content put the mix in a plastic condition. The gradations of the coarse and fine mixtures could be adjusted to try and meet SMA mix requirements. Adjustments in these two mixtures would result in mixtures similar to the intermediate gradation. The intermediate gradation resulted in a plastic mixture. It was deduced that a satisfactory SMA could not be made using the Franklin/Riley Co. aggregates. Excessive aggregate breakdown was suspected as the major cause.

### **Dolese Aggregates**

#### ***Preliminary Mix Designs***

The results from the preliminary mix designs for the Dolese aggregates for the coarse, intermediate and fine gradations are shown in Table 14. The results of the preliminary mix designs were performed at 6.5% asphalt for the coarse and intermediate mixtures and 7.0% for the fine mixture. The preliminary results for the coarse gradation showed high VTM, high VMA and low VFA, which indicates more asphalt is needed in the mixture. The Marshall stabilities were low, 4225 N, and the flows were high, 26, indicating too much

Table 14. Preliminary Mix Design Results, Dolese Aggregates.

Gradation	Asphalt Content (%)	Bulk Specific Gravity	Maximum Specific Gravity	Unit Weight (kN/m <sup>3</sup> )	VTM (%)	VMA (%)	VFA (%)	Marshall	
								Stability (N)	Flow (0.1 mm)
Coarse	6.5	2.208	2.405	21.66	8.2	20.4	59.8	4225	26
Intermediate	6.5	2.319	2.373	22.75	2.3	17.2	86.8	4360	20
Fine	7.0	2.296	2.363	22.53	2.8	18.6	84.9	4930	23

asphalt or excessive fine aggregate. Some aggregate breakdown was evident by observing the samples after Marshall stability testing.

Based on the conflicting test results and observations of coarse aggregate breakage, excessive aggregate breakdown was suspected. The high VMA indicated a need for a gradation closer to the maximum density line and/or more mineral filler. The intermediate gradation is the coarse gradation with more mineral filler and the fine gradation is closer to the maximum density line.

The mix properties of the coarse gradation were out of specification and the indicated changes in gradation were similar to the intermediate and fine gradations. However, due to suspected aggregate breakdown, additional samples were made to the coarse gradation at 6.0% and 7.0% asphalt to determine the influence of asphalt content on aggregate breakdown.

The preliminary results for the intermediate and fine gradations showed low VTM and high VFA, which indicates less asphalt is needed in the mixture. The VMA was acceptable. The Marshall stabilities were low (< 5000 N) and the flows were high (20-23), indicating too

much asphalt or excessive fine aggregate. The preliminary test results indicated a change in asphalt content could bring the mixtures into specification on VTM, VFA, Marshall stability and flow. Some aggregate breakdown was evident by observing the samples after Marshall stability testing.

### ***Coarse Gradation***

The mix design results for the coarse gradation are shown in Table 15 and graphically in Figures 8-13. The VTM and VMA values (Figures 8 & 9) were above the recommended range at all asphalt contents. The Marshall stabilities (Figure 12) were below the minimum recommended value (6200 N) and the flows (Figure 13) were well above the recommended maximum of 16, at all asphalt contents. The VFAs (Figure 9) were below recommended limits. A change in asphalt content had little to no effect on VTM or flow indicating possible excessive aggregate breakdown. An optimum asphalt content could not be determined and a suitable SMA mixture could not be made from the coarse gradation.

### ***Intermediate Gradation***

The mix design results for the intermediate gradation are shown in Table 15 and graphically in Figures 8-13. The VTMs (Figure 8) indicate an optimum asphalt content of approximately 5.5%. This is below NCAT's recommended minimum of 6.0% for adequate durability of SMA mixtures. The VMA (Figure 9) is also below the recommended minimum as well. The Marshall stabilities (Figure 12) were below the recommended minimums and the flows (Figure 13) above specification limits. The plot of VMA versus asphalt content (Figure 9)

Table 15. Mix Design Results, Dolese Aggregates.

Asphalt Content (%)	Bulk Specific Gravity	Maximum Specific Gravity	Unit Weight (kN/m^3)	VTM (%)	VMA (%)	VFA (%)	Marshall	
							Stability (N)	Flow (0.1 mm)
Coarse Gradation								
6.0	2.230	2.423	21.87	8.0	19.2	58.5	4280	23
6.5	2.208	2.405	21.66	8.2	20.4	59.8	4225	26
7.0	2.191	2.388	21.49	8.2	21.4	61.5	4070	22
Intermediate Gradation								
5.5	2.311	2.406	22.67	3.9	16.6	76.2	4890	24
6.0	2.317	2.397	22.72	3.4	16.8	80	4560	22
6.5	2.319	2.373	22.75	2.3	17.2	86.8	4360	20
Fine Gradation								
6.0	2.316	2.397	22.72	3.4	17.0	80.2	6375	26
6.5	2.324	2.380	22.80	2.3	17.2	86.4	6135	26
7.0	2.296	2.363	22.53	2.8	18.6	84.9	4925	23

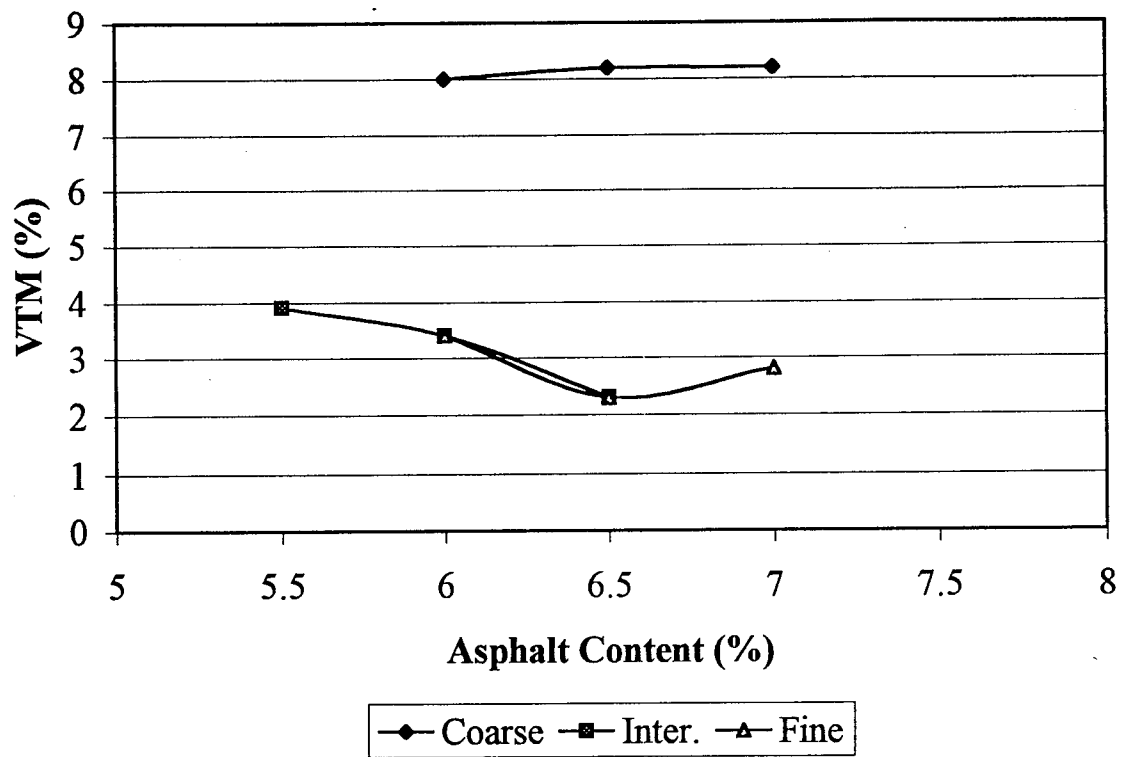


Figure 8. VTM vs. Asphalt Content, Dolese Aggregates.

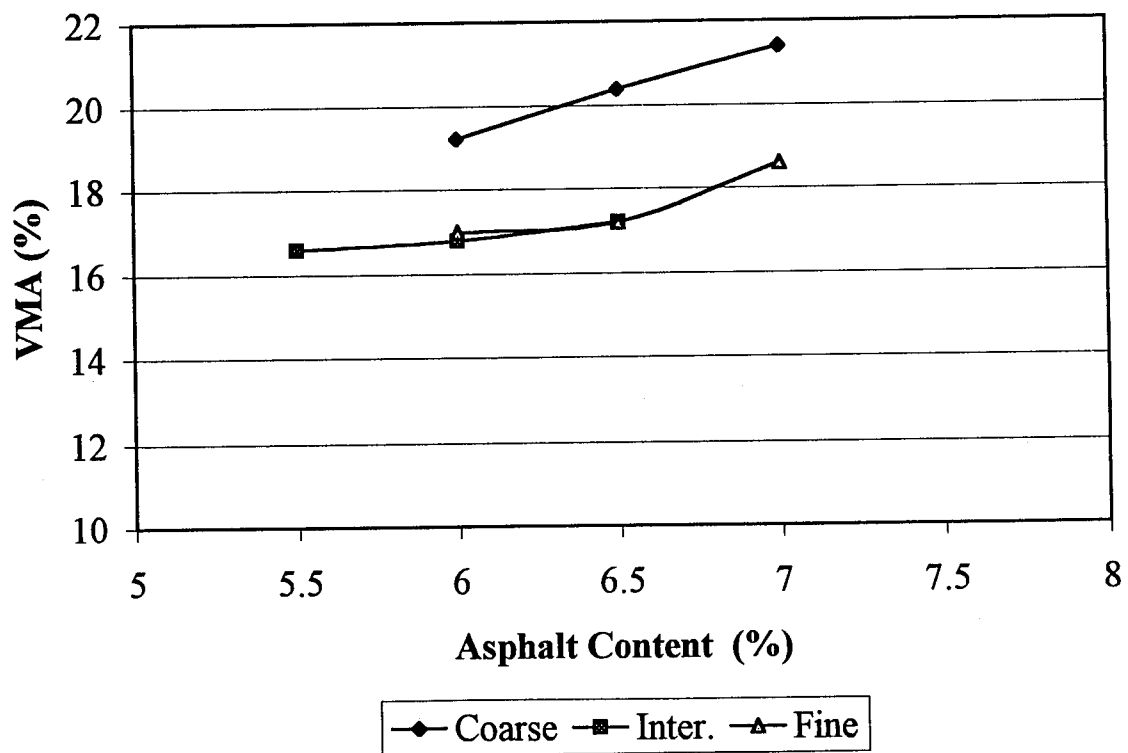


Figure 9. VMA vs. Asphalt Content, Dolese Aggregates.

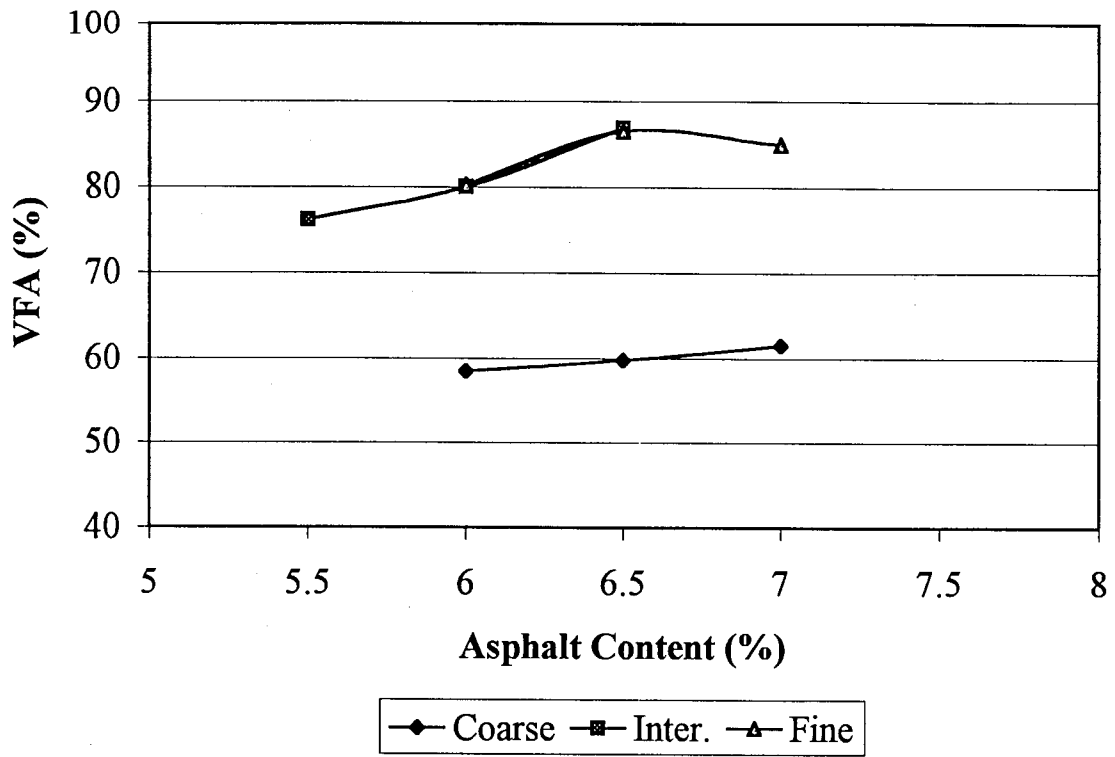


Figure 10. VFA vs. Asphalt Content, Dolese Aggregates.

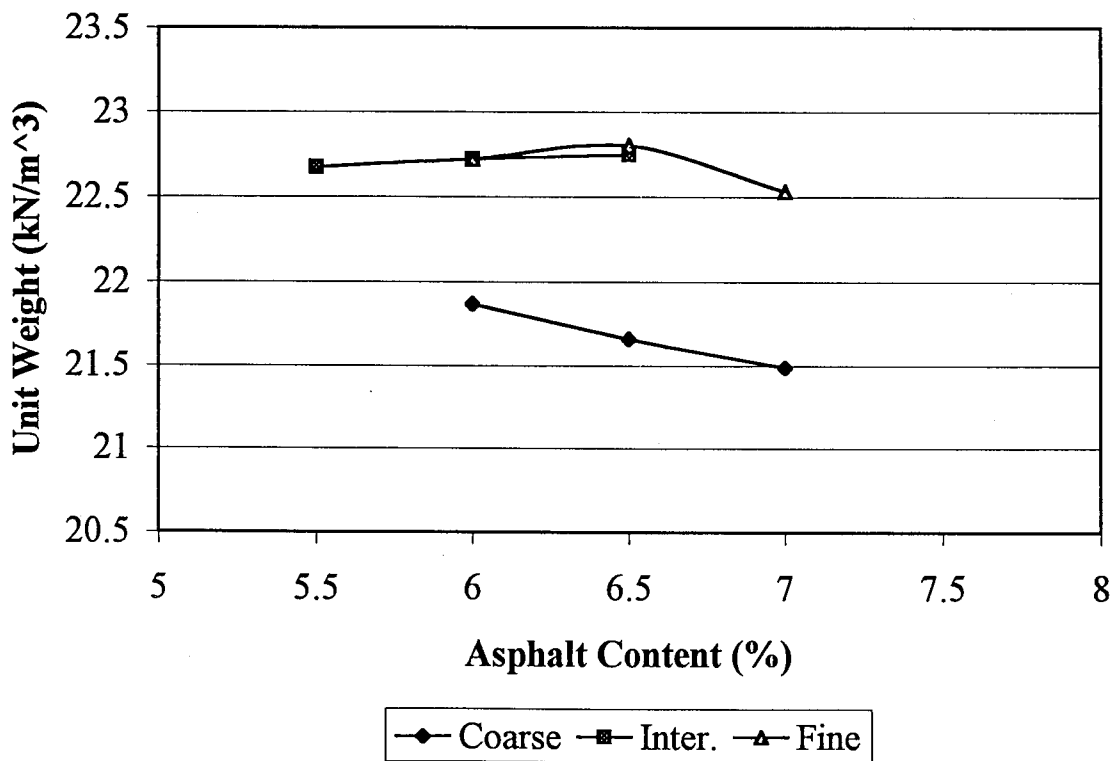


Figure 11. Unit Weight vs. Asphalt Content, Dolese Aggregates.

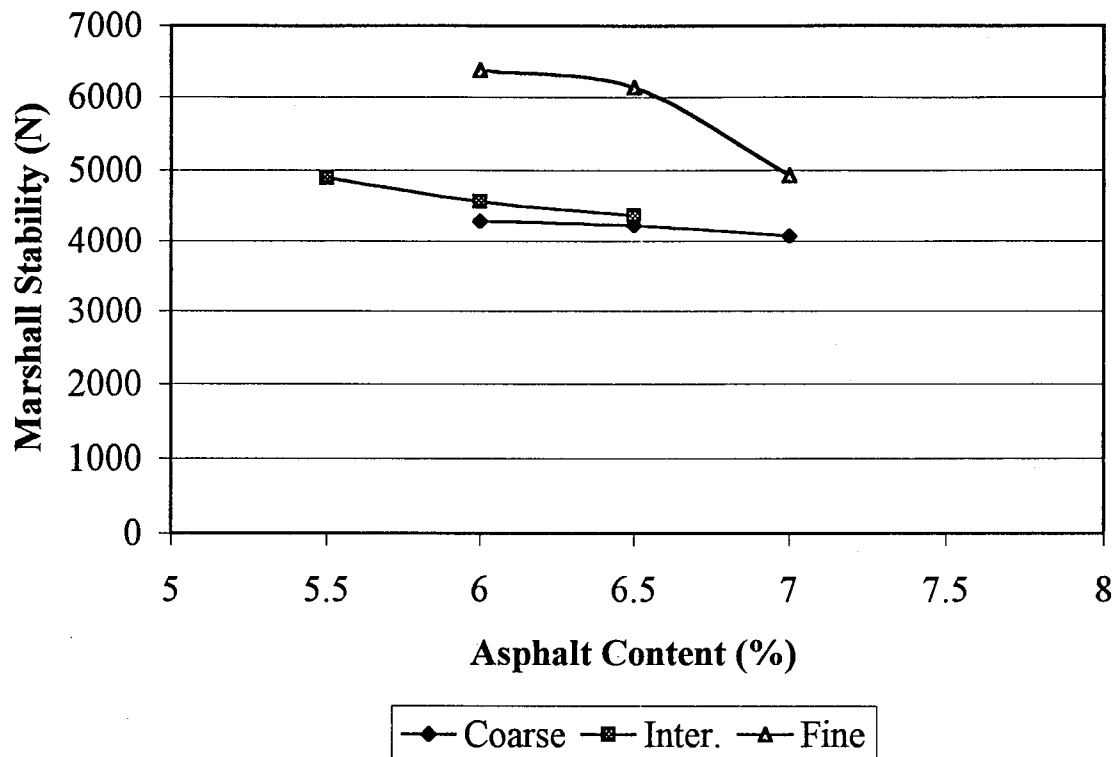


Figure 12. Marshall Stability vs. Asphalt Content, Dolese Aggregates.

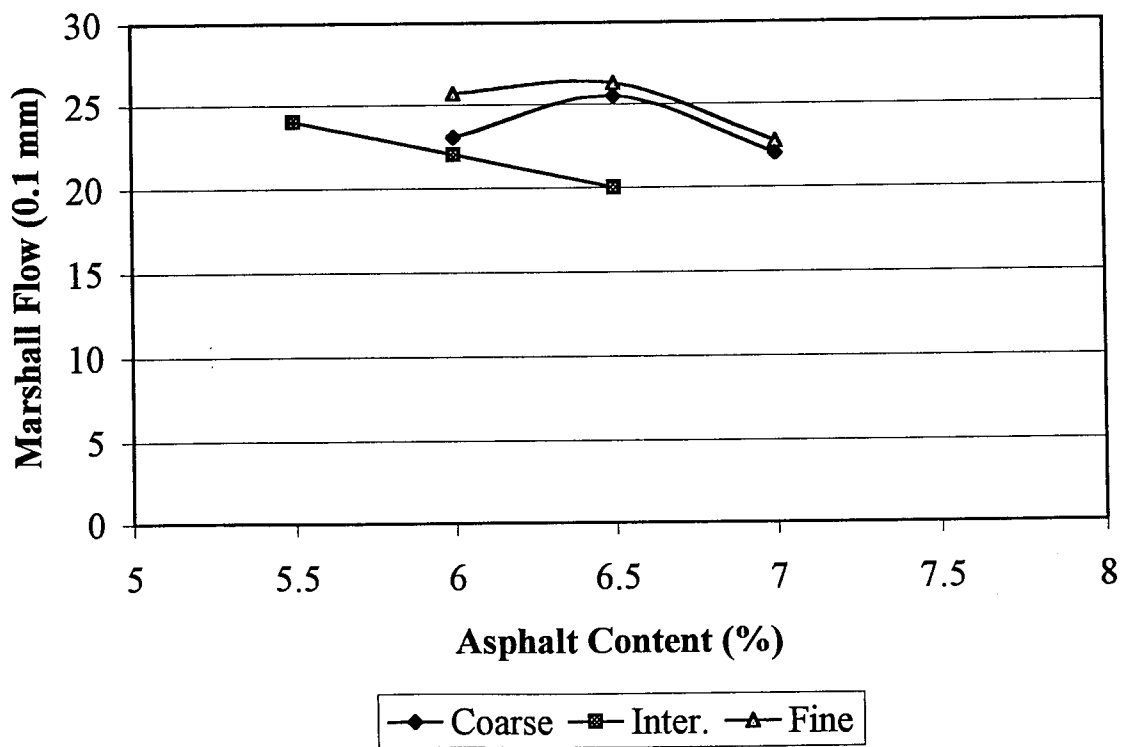


Figure 13. Marshall Flow vs. Asphalt Content, Dolese Aggregates.

shows that optimum asphalt content lies on the wet side of the VMA curve. The Asphalt Institute (13) recommends that mixtures not be designed on the wet side of the curve. On the wet side of the curve the asphalt cement is preventing good aggregate contact and the mixture is approaching a plastic condition. Coarsening the gradation to increase VMA would result in a mixture similar to the coarse gradation. A satisfactory mix meeting all the requirements could not be made with the intermediate gradation. Excess aggregate degradation was the suspected cause.

### ***Fine Gradation***

The results of the mix design for the fine gradation with the Dolese aggregates are shown in Table 15 and graphically in Figures 8-13 as well. As shown in Figure 8, reducing the asphalt content from 7.0% to 6.0% resulted in a decrease then an increase in VTM. The VMAs (Figure 9) went down with a reduction in asphalt content. The VFAs (Figure 10) were at the high end of the acceptable range. As shown in Figure 12, at the lowest asphalt content the Marshall stability (6370 N) was above the minimum (6200 N). All of the samples were above the recommended flow range (Figure 13). Figure 8 shows that optimum asphalt content, corresponding to 3.5% VTM would be approximately 5.8%, assuming a further reduction in asphalt would not lead to increased aggregate breakdown and a reduction or no change in VTM as seen with the other Dolese gradations. As with the intermediate mix, the optimum asphalt content was on the wet side or near the minimum VMA (Figure 9). Aggregate breakdown was visible in the Marshall stability samples.

## Summary

None of the Dolese aggregate mixtures met all the requirements for an SMA mixture. The gradations of the mixtures could be adjusted to try and meet SMA mix requirements. However, the behavior of these mixtures with a change in asphalt content was erratic indicating possible excessive aggregate breakdown. Aggregate breakdown was investigated under Task 4.

## Formoso Aggregates

The Dolese and Franklin/Riley Co. aggregates showed signs of excessive aggregate breakdown during mix design. With the soft nature of the Formoso aggregates, it was decided to try a preliminary mix design using the fine gradation only. It was believed that the coarse and intermediate gradations would result in even more aggregate breakdown. The results of the preliminary fine gradation using the Formoso CS-1 with the chat and Dolese CS-2 and MFS-2 are shown in Table 16.

Table 16. Preliminary Mix Design Results, Formosa, KS Aggregates.

Asphalt Content (%)	Bulk Specific Gravity	Maximum Specific Gravity	Unit Weight (kN/m <sup>3</sup> )	VTM (%)	VMA (%)	VFA (%)	Marshall	
							Stability (N)	Flow (0.1 mm)
7.0	2.075	2.167	20.36	4.2	14.7	71.0	3450	18

The Formoso, KS aggregates did not meet the minimum specification requirements for either an SMA or hot mix asphalt (HMA) mixture. The results in Table 16 show that an acceptable SMA mixture could not be made. The VMA was low (14.7%), the Marshall stability was low (3450 N) and the flow was high. However, the flow value of 18 was the lowest recorded of any of the SMA mixtures tested. The VTM was near 4.0% and the VFA was within an acceptable range. Considerable aggregate breakdown was observed in the samples both prior to and after Marshall stability testing. With the excessive aggregate breakdown observed, it was obvious that a suitable HMA or SMA mixture could not be made with the Formosa, KS CS-1. No further performance testing was performed on the Formosa aggregates.

## **CHAPTER 5**

### **AGGREGATE DEGRADATION**

#### **(Task 4)**

One of the major concerns in using SMA in Kansas has been the limited supply of aggregates with a LA Abrasion less than 35%, the usual recommended maximum (8). To investigate aggregate degradation, the Marshall samples were tested for gradation after compaction and stability and flow testing. The gradation was compared to the Job Mix Formula (JMF) to determine aggregate degradation and the amount of degradation was compared to LA Abrasion to see if any correlation exists.

The percent aggregate degradation was determined by comparing the percent material retained on each individual sieve as batched (from the JMF) and after compaction and testing. The sum of the percent retained on each individual sieve and in the pan would equal 100%. Aggregate degradation was quantified by subtracting the individual percent retained from the JMF from the individual percent retained after testing. A negative number indicates a reduction in material from the JMF and a positive number an increase on the respective sieve.

#### **COARSE SMA GRADATION**

The Franklin/Riley Co. aggregates and Dolese aggregates were used to make the coarse SMA mixtures. The average results of the recovered gradation analysis for the coarse gradation SMA mixtures are shown in Table 17 along with the percent retained on each

Table 17. Aggregate Degradation, Coarse SMA Gradation.

Sieve Size (mm)	Coarse Gradation JMF		Franklin/Riley Co.		Dolese	
	Percent Retained	% Retained & Passing	Percent Retained	% Retained & Passing	Aggregate Degradation	Percent Retained & Passing
19.0	0	0	0	0	0.0	0
12.5	13.5	13.5	9.2	9.2	-4.3	12.2
9.5	45.0	31.5	30.6	21.4	-10.1	23.1
4.75	82.0	37.0	66.6	36.0	-1.0	35.7
2.36	86.0	4.0	78.8	12.2	8.2	10.6
1.18	88.0	2.0	83.1	4.3	2.3	3.4
0.600	89.0	1.0	85.4	2.3	1.3	1.8
0.300	90.0	1.0	86.9	1.5	0.5	1.1
0.150	92.2	2.2	88.9	2.0	-0.2	1.5
0.075	93.9	1.7	91.1	2.2	0.5	1.9
Pan	100	6.1	100	8.9	2.8	8.7
						2.6

individual sieve and the percent degradation. The percent degradation is presented graphically in Figure 14.

The results indicate a substantial aggregate breakdown with more breakdown in the softer aggregates (Franklin/Riley Co. aggregates). The excessive aggregate breakdown results in a considerably different gradation than originally batched. Due to the fractured pieces of aggregate, many uncoated pieces of aggregate were noted as well. KDOT's specification (2) for SMA gradation control indicates a tolerance of 4 percent on the 4.75 mm to 1.18 mm sieves. The maximum amount of aggregate breakdown occurred on the 9.5 mm sieve, 10.1% and 8.4% for the Franklin/Riley Co. and Dolese aggregates, respectively. There is no current gradation tolerance specification on the 9.5 mm sieve. However, the breakdown of the plus 9.5 mm aggregate resulted in an increase in the materials retained on the 2.36 mm sieve of 8.2% and 6.6%, respectively. There was little change in the 4.75 mm material, which was almost entirely chat. Due to the increase in material on the 2.36 mm sieve from the breakdown of the plus 9.5 mm material, the specification limit would be hard to meet. It is believed that this excessive aggregate breakdown, mainly on the 12.5 and 9.5 mm sieves, led to the inability to make a satisfactory SMA mixture.

## **INTERMEDIATE SMA GRADATION**

The average results of the recovered gradation analysis for the intermediate gradation SMA mixtures are shown in Tables 18 and 19, along with the percent retained on each individual sieve and the percent degradation, for the Franklin/Riley Co. and Dolese aggregates,

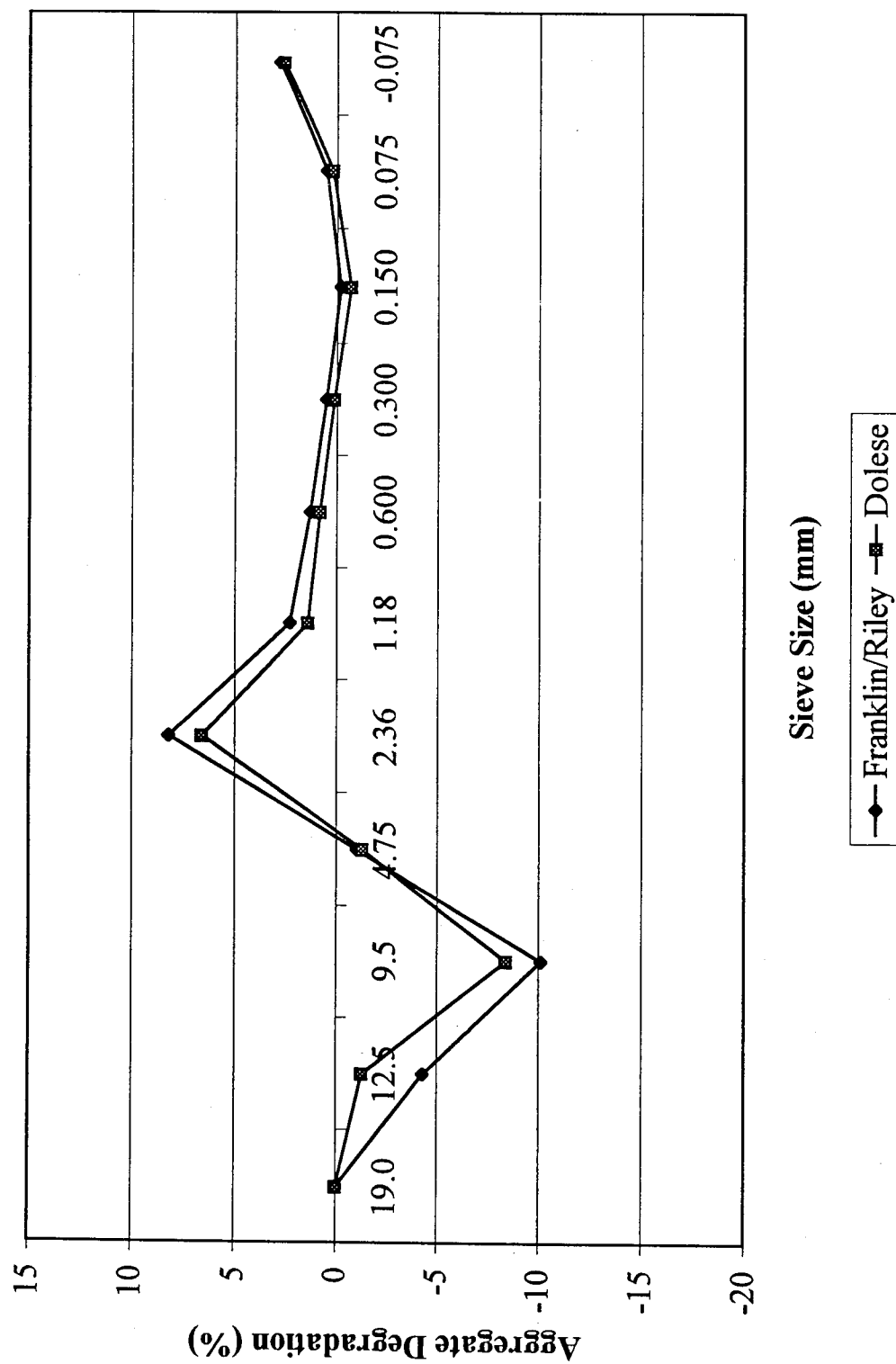


Figure 14. Aggregate Degradation, Coarse SMA Gradation.

Table 18. Aggregate Degradation Intermediate SMA Gradation, Franklin/Riley Co. Aggregates.

Sieve Size (mm)	Inter. Gradation JMF		Percent Retained			% Retained & Passing			Aggregate Degradation (%)
	Percent Retained	% Retained & Passing	6.0% AC	6.5% AC	7.0% AC	6.0% AC	6.5% AC	7.0% AC	
19	0	0	0	0	0	0	0	0	0
12.5	13.5	13.5	6.7	9.6	11.4	6.7	9.6	11.4	-4.3
9.5	39.8	26.3	26.1	25.8	32.8	19.4	16.2	21.4	-7.3
4.75	76.4	36.6	60.9	58.3	66.1	34.8	32.5	33.3	-3.1
2.36	80.4	4.0	71.2	70.2	77.5	10.3	11.9	11.4	7.2
1.18	81.7	1.3	75.1	74.0	80.6	3.9	3.8	3.1	2.3
0.600	82.3	0.6	79.5	78.1	82.3	4.4	4.1	1.7	2.8
0.300	83.0	0.7	81.4	80.0	82.4	1.9	1.9	0.1	0.6
0.150	86.7	3.7	84.7	83.5	86.1	3.3	3.5	3.7	-0.2
0.075	89.6	2.9	88.2	87.0	89.0	3.5	3.5	2.9	0.4
Pan	100	10.4	100	100	100	11.8	13	11	1.5

Table 19. Aggregate Degradation Intermediate SMA Gradation, Dolse Aggregates.

Sieve Size (mm)	Inter. Gradation JMF		Percent Retained			% Retained & Passing			Aggregate Degradation (%)
	Percent Retained	% Retained & Passing	5.5% AC	6.0% AC	6.5% AC	5.5% AC	6.0% AC	6.5% AC	
19.0	0	0	0	0	0	0	0	0	0
12.5	13.5	13.5	11.1	11.4	11.5	11.1	11.4	11.5	-2.2
9.5	39.8	26.3	32.8	32.8	30.5	21.7	21.4	19.0	-5.6
4.75	76.4	36.6	66.3	66.1	66.2	33.5	33.3	35.7	-2.4
2.36	80.4	4.0	77.0	77.5	76.6	10.7	11.4	10.4	6.8
1.18	81.7	1.3	79.9	80.6	79.7	2.9	3.1	3.1	1.7
0.600	82.3	0.6	81.4	82.3	81.4	1.5	1.7	1.7	1.0
0.300	83.0	0.7	82.2	82.4	82.5	0.8	0.1	1.1	0.0
0.150	86.7	3.7	83.6	86.1	84.5	1.4	3.7	2.0	-1.3
0.075	89.6	2.9	86.2	89.0	87.4	2.6	2.9	2.9	-0.1
Pan	100	10.4	100	100	100	13.8	11	12.6	2.1

respectively. The average percent degradation for each aggregate source is shown graphically in Figure 15.

The results indicate substantial aggregate breakdown with more breakdown in the softer aggregate (Franklin/Riley Co. aggregates). As with the coarse gradation the maximum aggregate breakdown occurred on the 9.5 mm sieve. There was minor breakdown on the 4.75 mm sieve, which is composed almost entirely of chat. The aggregate breakdown on the plus 4.75 mm sieves resulted in an increase in material of 7.2% and 8.8% on the 2.36 mm sieve for the Franklin/Riley Co. and Dolese aggregates, respectively. Due to aggregate breakdown on the plus 4.75 mm sieves, meeting the specification limit of 4% on the 2.36 mm sieve would be difficult. It is believed that this excessive aggregate breakdown, mainly on the 12.5 and 9.5 mm sieves led to the inability to make a satisfactory SMA mixture.

## **FINE SMA GRADATION**

The average results of the recovered gradation analysis for the fine gradation SMA mixtures are shown in Tables 20-22, along with the percent retained on each individual sieve and the percent degradation, for the Franklin/Riley Co., Dolese and Formosa aggregates, respectively. The average percent degradation for each aggregate source is shown graphically in Figure 16.

The results indicate substantial aggregate breakdown with more breakdown in the softer aggregates (Formosa aggregates). The excessive aggregate breakdown results in a considerably different gradation than originally batched. Due to the fractured pieces of

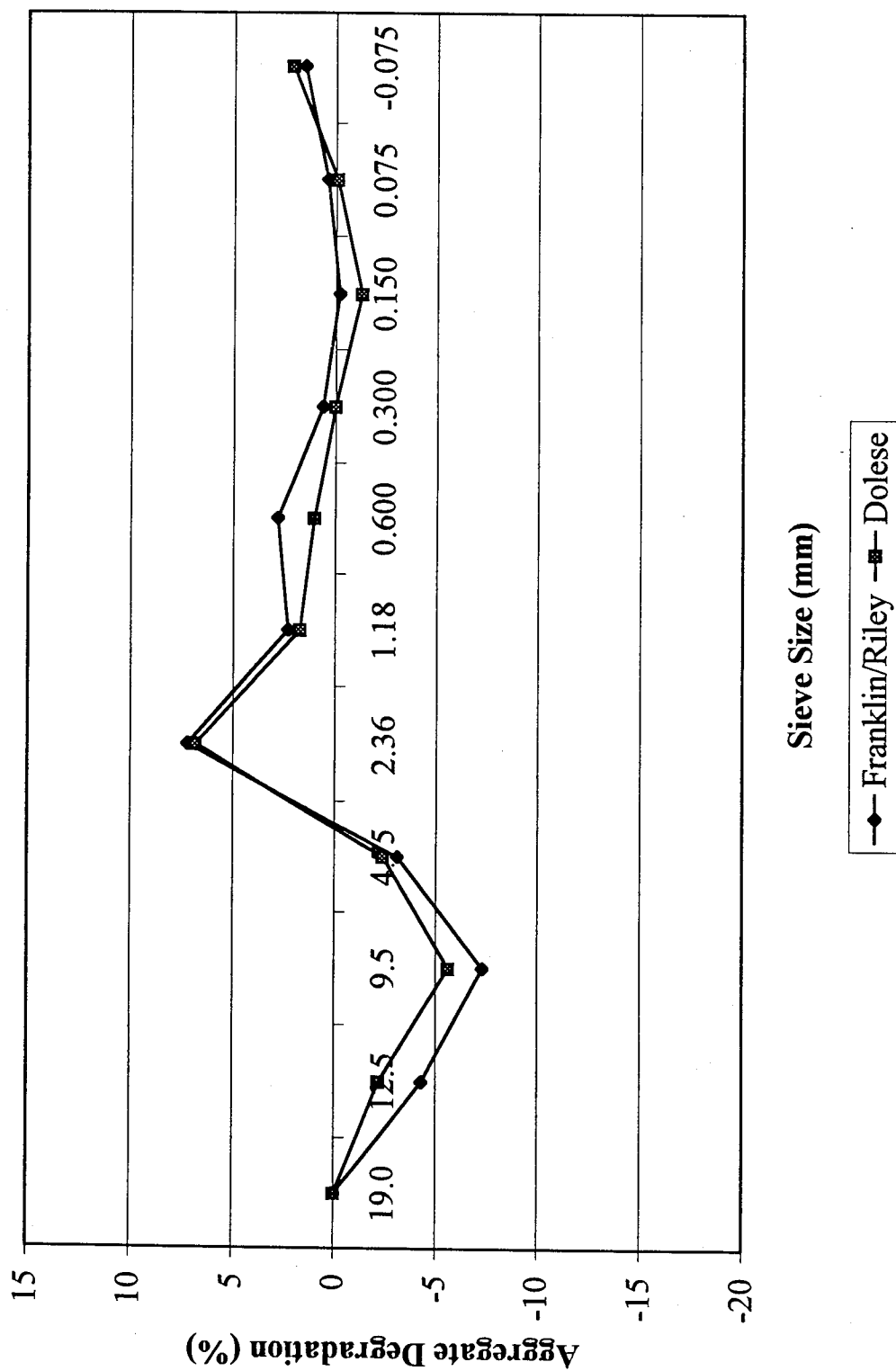


Figure 15. Aggregate Degradation, Intermediate SMA Gradation.

Table 20. Aggregate Degradation, SMA Fine Gradation, Franklin/Riley Co. Aggregates.

Sieve Size (mm)	Fine Gradation JMF		6.0% AC		6.5% AC		7.0% AC		6.0% AC		6.5% AC		7.0% AC		Aggregate Degradation (%)	
	Percent Retained	(%) Retained & Passing	Percent Retained		Percent Retained		Percent Retained		Percent Retained & Passing		Percent Retained & Passing		Percent Retained & Passing		Percent Retained & Passing	
19.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.0
12.5	7.0	7.0	5.6	6.0	5.6	5.6	5.6	5.6	5.6	6	5.6	5.6	5.6	5.6	-1.3	-1.3
9.5	29.8	22.8	19.6	21.3	20.2	20.2	20.2	20.2	14.0	15.3	14.6	14.6	14.6	14.6	-8.2	-8.2
4.75	69.8	40.0	60.7	61.9	62.0	62.0	62.0	62.0	41.1	40.6	41.8	41.8	41.8	41.8	1.2	1.2
2.36	75.0	5.2	70.7	72.0	71.5	71.5	71.5	71.5	10.0	10.1	9.5	9.5	9.5	9.5	4.7	4.7
1.18	77.1	2.1	74.7	75.3	74.7	74.7	74.7	74.7	4.0	3.3	3.2	3.2	3.2	3.2	1.4	1.4
0.600	81.4	4.3	78.3	78.6	78	78	78	78	3.6	3.3	3.3	3.3	3.3	3.3	-0.9	-0.9
0.300	82.2	0.8	80.5	80.3	79.5	79.5	79.5	79.5	2.2	1.7	1.5	1.5	1.5	1.5	1.0	1.0
0.150	86.7	4.5	85.0	83.7	83.4	83.4	83.4	83.4	4.5	3.4	3.9	3.9	3.9	3.9	-0.6	-0.6
0.075	89.6	2.9	89.1	87.2	87.1	87.1	87.1	87.1	4.1	3.5	3.7	3.7	3.7	3.7	0.9	0.9
Pan	100	10.4	100	100	100	100	100	100	10.9	12.8	12.9	12.9	12.9	12.9	1.8	1.8

Table 21. Aggregate Degradation, SMA Fine Gradation, Dolese Aggregates.

Sieve Size (mm)	Fine Gradation JMF		6.0% AC		6.5% AC		7.0% AC		6.0% AC		6.5% AC		7.0% AC		Aggregate Degradation (%)	
	Percent Retained	% Retained & Passing	Percent Retained		Percent Retained		Percent Retained		Percent Retained & Passing		Percent Retained & Passing		Percent Retained & Passing		Percent Retained & Passing	
19.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12.5	7.0	7.0	6.7	6.6	6.6	6.6	6.6	6.7	6.6	6.6	6.6	6.6	6.6	6.6	6.6	-0.4
9.5	29.8	22.8	23.4	21.4	21.4	28.1	28.1	16.7	14.8	14.8	21.5	21.5	21.5	21.5	21.5	-5.1
4.75	69.8	40.0	64.7	65	65	65.2	65.2	41.3	43.6	43.6	37.1	37.1	37.1	37.1	37.1	0.7
2.36	75.0	5.2	71.9	72.7	72.7	73.3	73.3	7.2	7.7	7.7	8.1	8.1	8.1	8.1	8.1	2.5
1.18	77.1	2.1	74.2	75.0	75.0	75.5	75.5	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	0.2
0.600	81.4	4.3	77.3	78.1	78.1	78.4	78.4	3.1	3.1	3.1	2.9	2.9	2.9	2.9	2.9	-1.3
0.300	82.2	0.8	79.2	79.8	79.8	80.1	80.1	1.9	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.0
0.150	86.7	4.5	82.8	83.4	83.4	83.7	83.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	-0.9
0.075	89.6	2.9	86.2	87.0	87.0	87.2	87.2	3.4	3.6	3.6	3.5	3.5	3.5	3.5	3.5	0.6
Pan	100	10.4	100	100	100	100	100	13.8	13	13	12.8	12.8	12.8	12.8	12.8	2.8

Table 22. Aggregate Degradation, SMA Fine Gradation, Formosa Aggregates.

Sieve Size (mm)	Fine Gradation JMF		Formosa, KS		Aggregate Degradation (%)
	Percent Retained	(%) Retained & Passing	Percent Retained	% Retained & Passing	
19.0	0	0	0	0	0
12.5	7.0	7.0	2.4	2.4	-4.6
9.5	29.8	22.8	9.1	6.7	-16.1
4.75	69.8	40.0	52.0	42.9	2.9
2.36	75.0	5.2	67.0	15.0	9.8
1.18	77.1	2.1	72.0	5.0	2.9
0.600	81.4	4.3	76.6	4.6	0.3
0.300	82.2	0.8	79.6	3.0	2.2
0.150	86.7	4.5	81.2	1.6	-2.9
0.075	89.6	2.9	83.9	2.7	-0.2
Pan	100	10.4	100.0	16.1	5.7

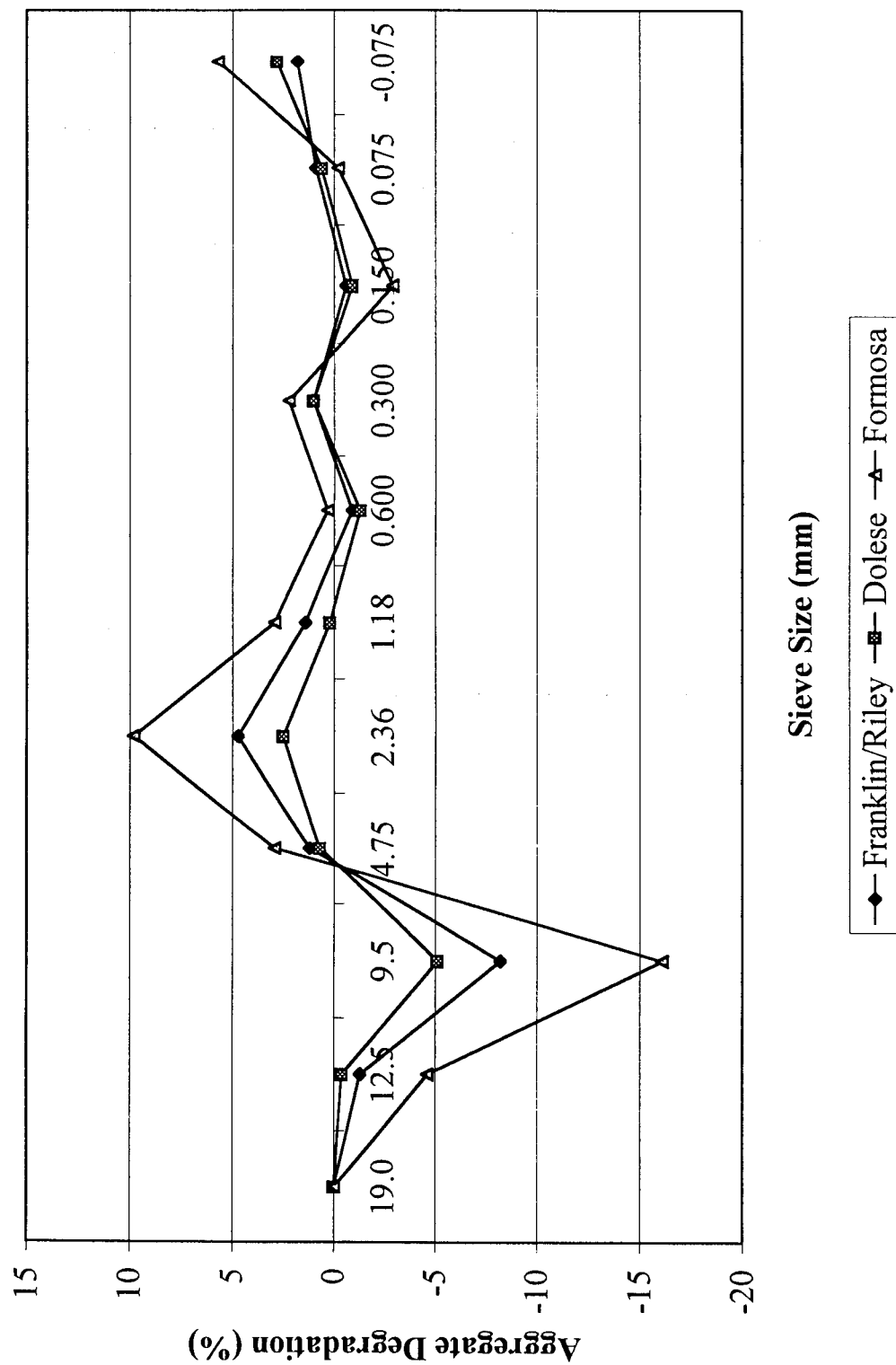


Figure 16. Aggregate Degradation, Fine SMA Gradation.

aggregate, many uncoated pieces of aggregate were noted as well. Again, all of the aggregate breakdown occurred on the plus 9.5 mm material. The Formosa aggregate showed considerable amounts of breakdown (16.1%) compared to the Franklin/Riley Co. aggregates ( 8.2%) and the Dolese (5.1%) aggregates. The aggregate breakdown led to an increase in material on the 2.36 mm sieve of 4.7%, 2.5% and 9.8% for the Franklin/Riley Co., Dolese and Formosa aggregates, respectively. The aggregate breakdown would make it to meet the tolerance limits on the 2.36 mm sieve for all but the Dolese aggregates. Little breakdown occurred in the chat.

#### **Without Primary Aggregate**

The above results indicated little to no aggregate breakdown in the chat. To investigate the effect of the chat on aggregate degradation, the chat was replaced with an equivalent amount and size of Franklin/Riley Co. limestone. The fine SMA gradation was used because it had shown the least amount of aggregate breakdown. The mix design for the Franklin/Riley Co. aggregates without chat are shown in Table 23 and graphically in Figures 17-22.

Table 23. Mix Design Results, Franklin/Riley Co. Aggregates, no Chat.

Asphalt Content (%)	Bulk Specific Gravity	Maximum Specific Gravity	Unit Weight (kN/m <sup>3</sup> )	VTM (%)	VMA (%)	VFA (%)	Marshall	
							Stability (N)	Flow (0.1 mm)
5.85	2.248	2.363	22.05	4.9	17.2	71.6	6225	27
6.35	2.234	2.347	21.94	4.8	18.1	73.5	5615	27
6.85	2.255	2.331	21.97	3.3	18.8	82.4	5060	23

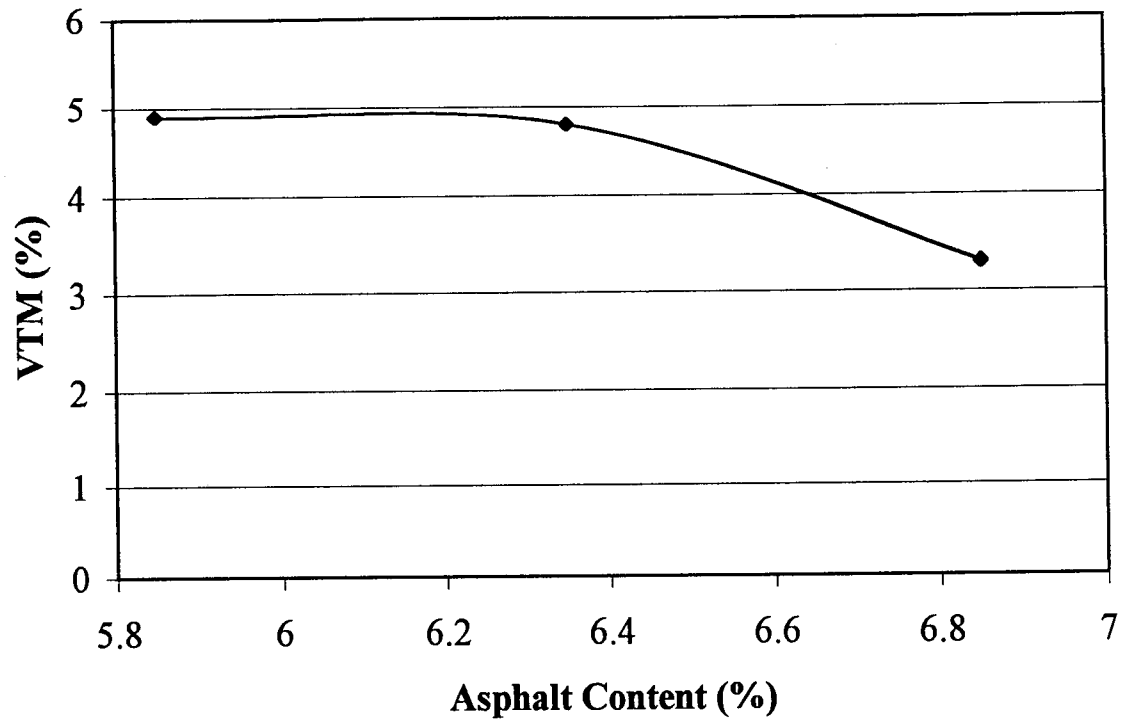


Figure 17. VTM vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.

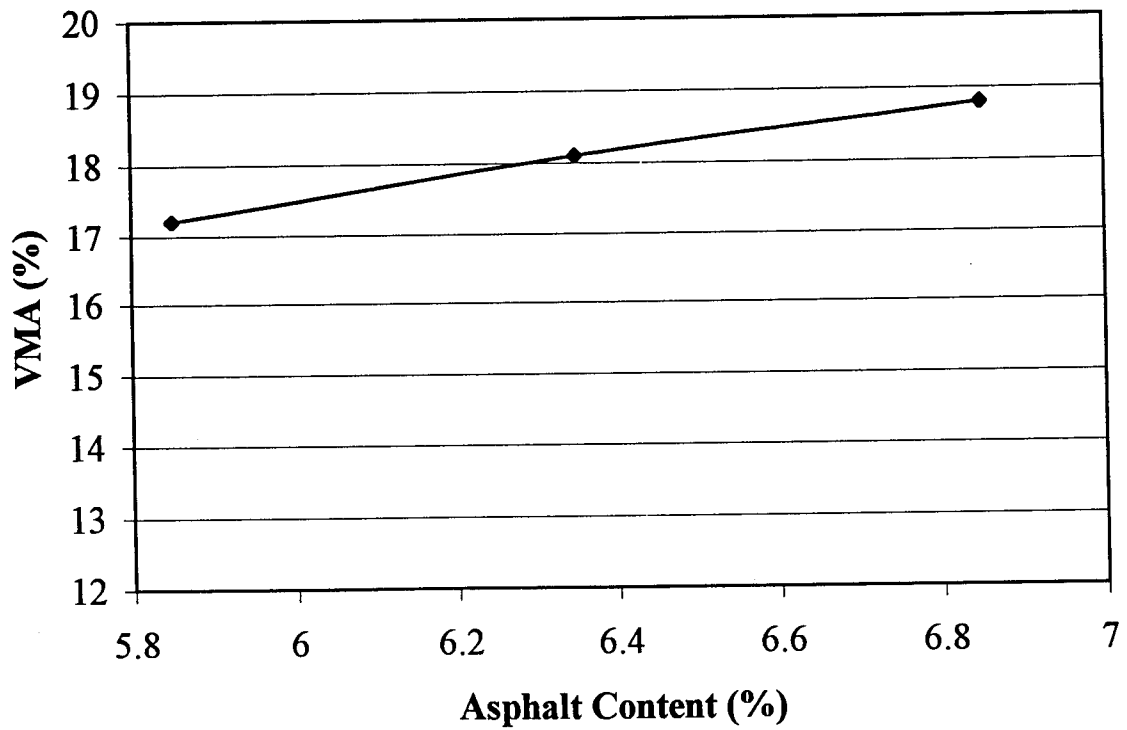


Figure 18. VMA vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.

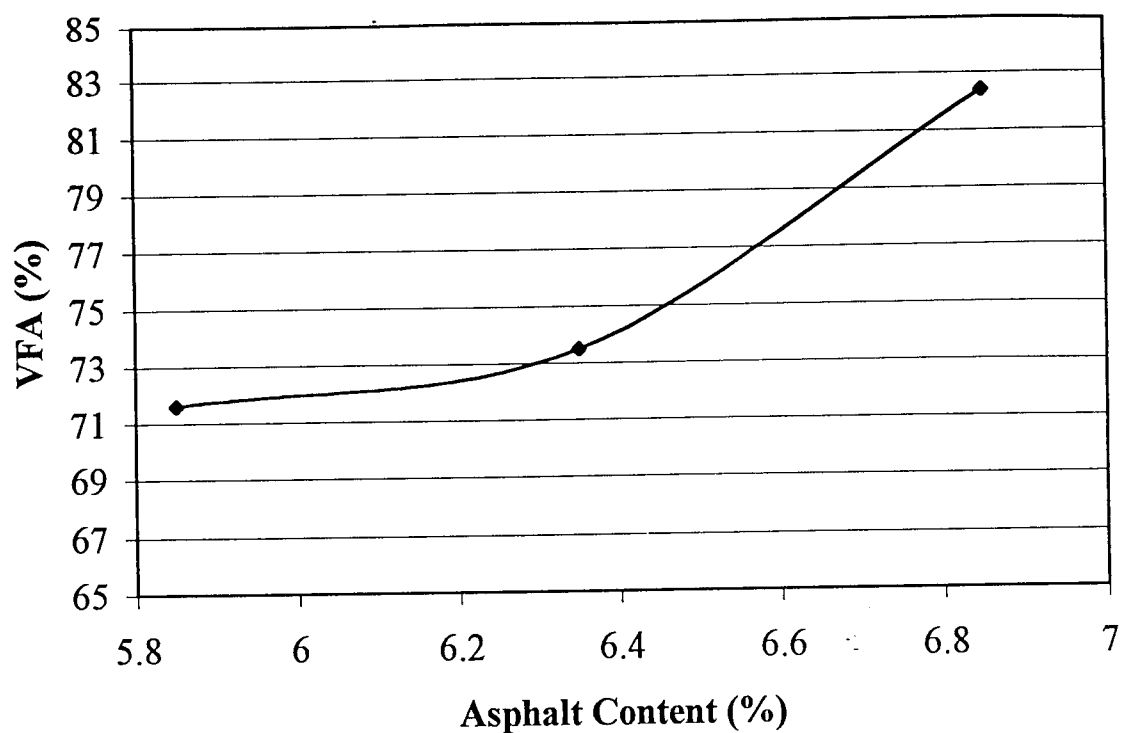


Figure 19. VFA vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.

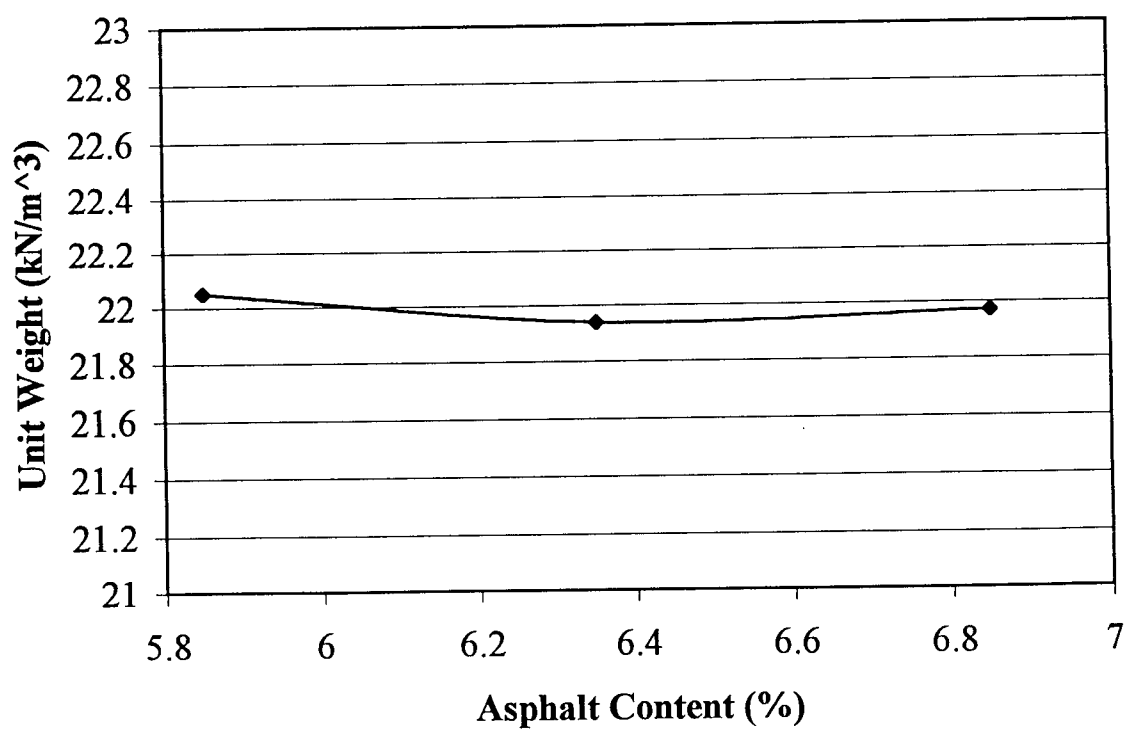


Figure 20. Unit Weight vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.

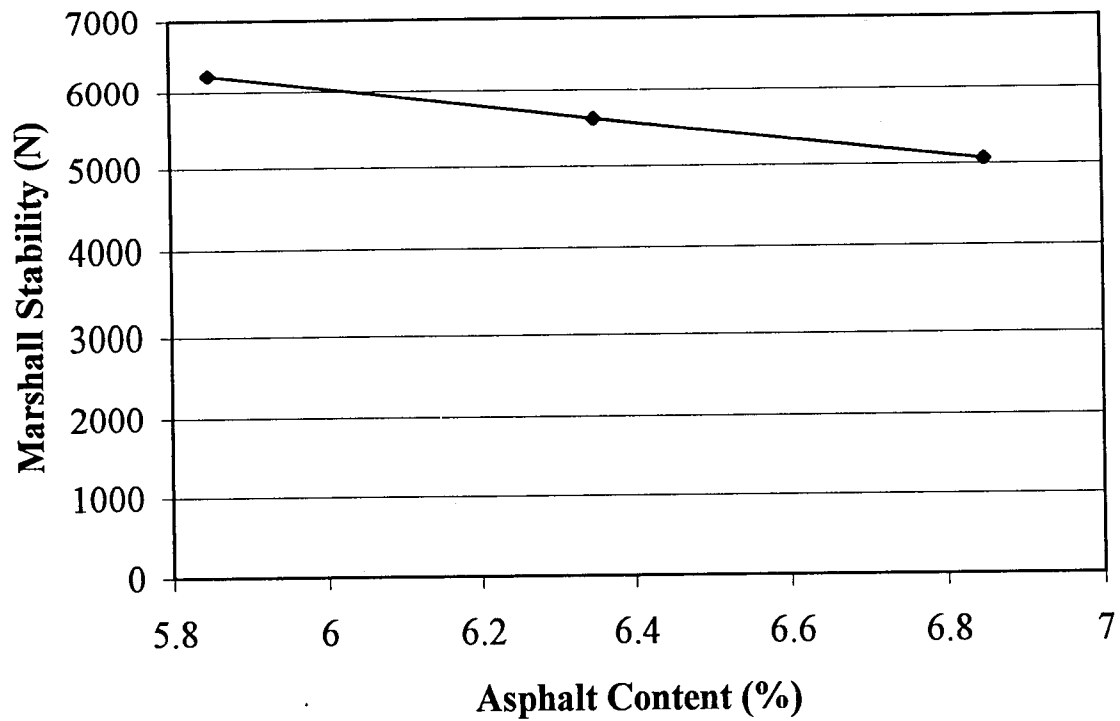


Figure 21. Marshall Stability vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.

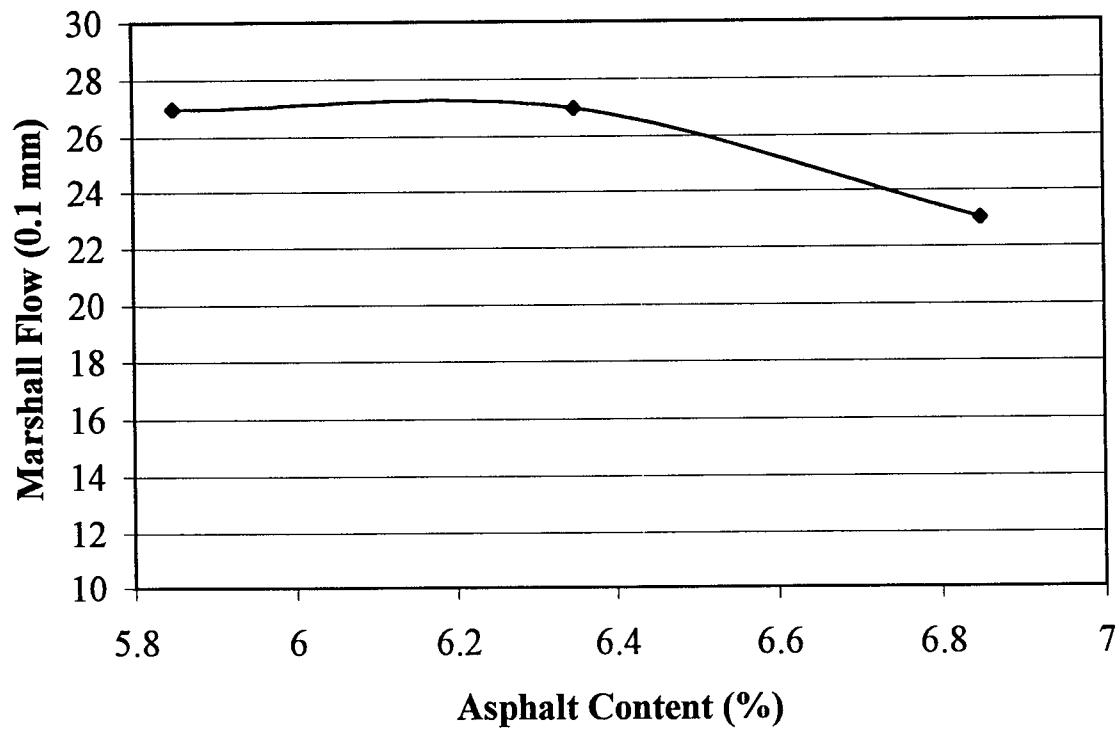


Figure 22. Marshall Flow vs. Asphalt Content, Franklin/Riley Co. Aggregates, no Chat.

The average results of the recovered gradation analysis for the fine gradation SMA mixtures using the Franklin/Riley Co. aggregates are shown in Table 24, along with the percent retained and passing each individual sieve and the percent degradation. The average percent degradation, with and without chat, is shown graphically in Figure 23.

The results in Figure 23 show that removing the chat resulted in a slight decrease in aggregate breakdown on the 9.5 mm sieve ( 8.2 to 3.6) but a substantial increase on the 4.75 mm sieve (0 to 6.3%). By summing the amount of aggregate breakdown the effect, if any, of the chat on aggregate breakdown can be determined. The total amount of aggregate breakdown with chat was 9.5%. The total amount of breakdown without chat was 10.1%, a difference of 0.6%. Removing the chat simply shifted the curve to the right but had no effect on total aggregate breakdown. The mix without chat still exceeded the specification tolerance on the 2.36 mm sieve as well as on the 4.75 mm sieve.

#### **LA ABRASION vs. AGGREGATE DEGRADATION**

One of the objectives of this study was to determine if there was a relationship between a specification property such as LA Abrasion and aggregate degradation and to see if a threshold limit of LA Abrasion could be established for Kansas aggregates in SMA mixtures. The majority of aggregate breakdown occurred on the 9.5 mm sieve, therefore, the 9.5 mm sieve was selected to quantify degradation. The aggregate breakdown has been shown as a negative number in the previous Tables, the sign has been switched in the following figures for clarity.

Table 24. Aggregate Degradation Fine SMA Gradation, Franklin/Riley Co. Aggregates, no Chat.

Sieve Size (mm)	Fine Gradation JMF		Aggregate Degradation			
	Percent Retained	% Retained & Passing	Percent Retained		Percent Retained & Passing	
			5.85% AC	6.35% AC	6.85% AC	Aggregate Degradation (%)
19.0	0	0	0	0	0	0.0
12.5	7.0	7.0	9.6	7.7	6.8	1.0
9.5	29.8	22.8	25.8	30.8	23.1	-3.6
4.75	69.8	40.0	58.3	65.7	34.9	-6.3
2.36	75.0	5.2	70.2	73.0	7.3	4.9
1.18	77.1	2.1	73.9	75.5	2.5	1.1
0.600	81.4	4.3	78.1	78.9	3.4	-0.4
0.300	82.2	0.8	80.0	80.4	1.5	1.1
0.150	86.7	4.5	83.4	83.3	2.9	-1.4
0.075	89.6	2.9	87.0	86.5	3.2	0.5
Pan	100	10.4	100	100	13.5	3.0

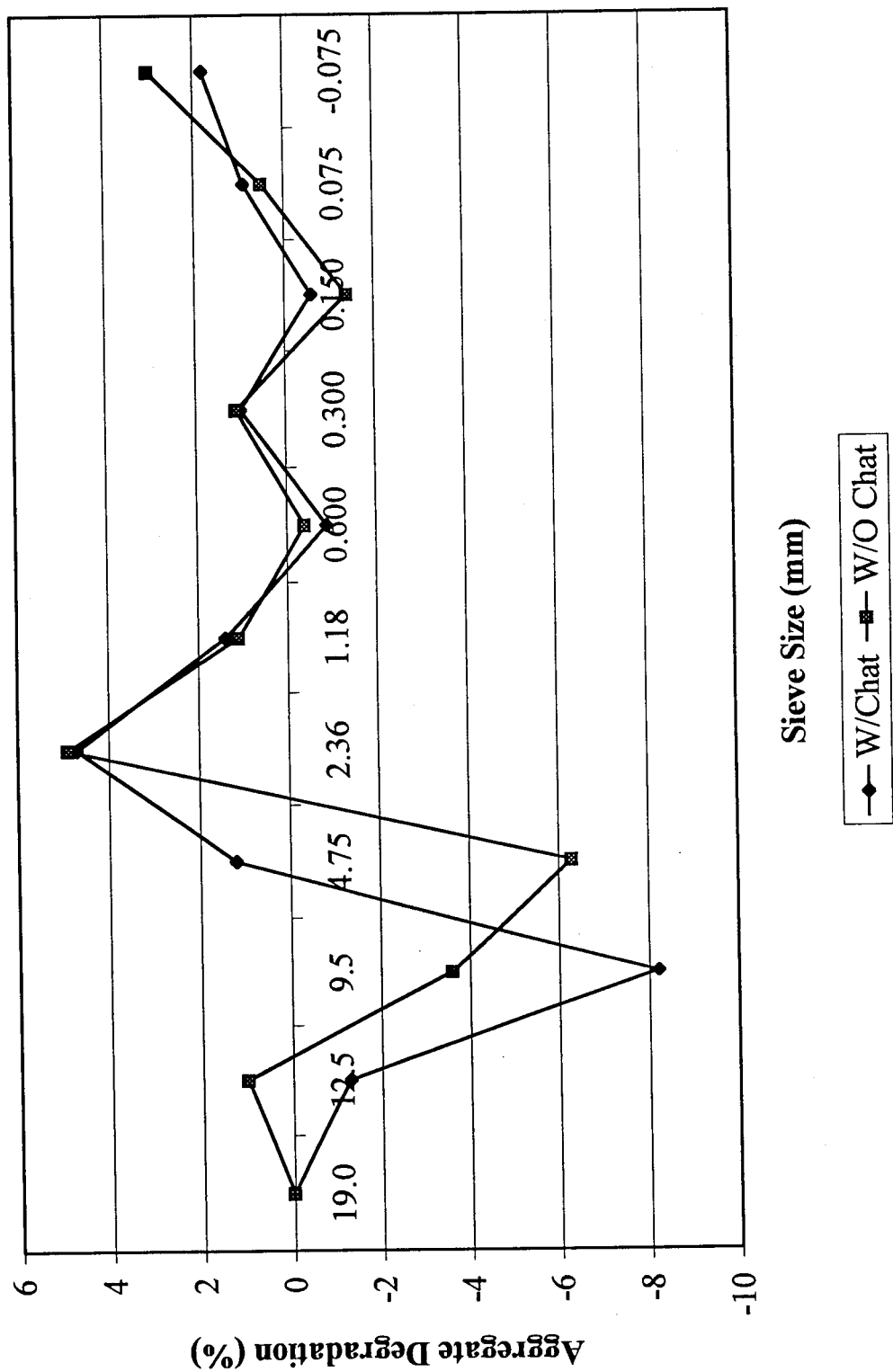


Figure 23. Effect of Chat on Aggregate Degradation, Franklin/Riley Co. Aggregates.

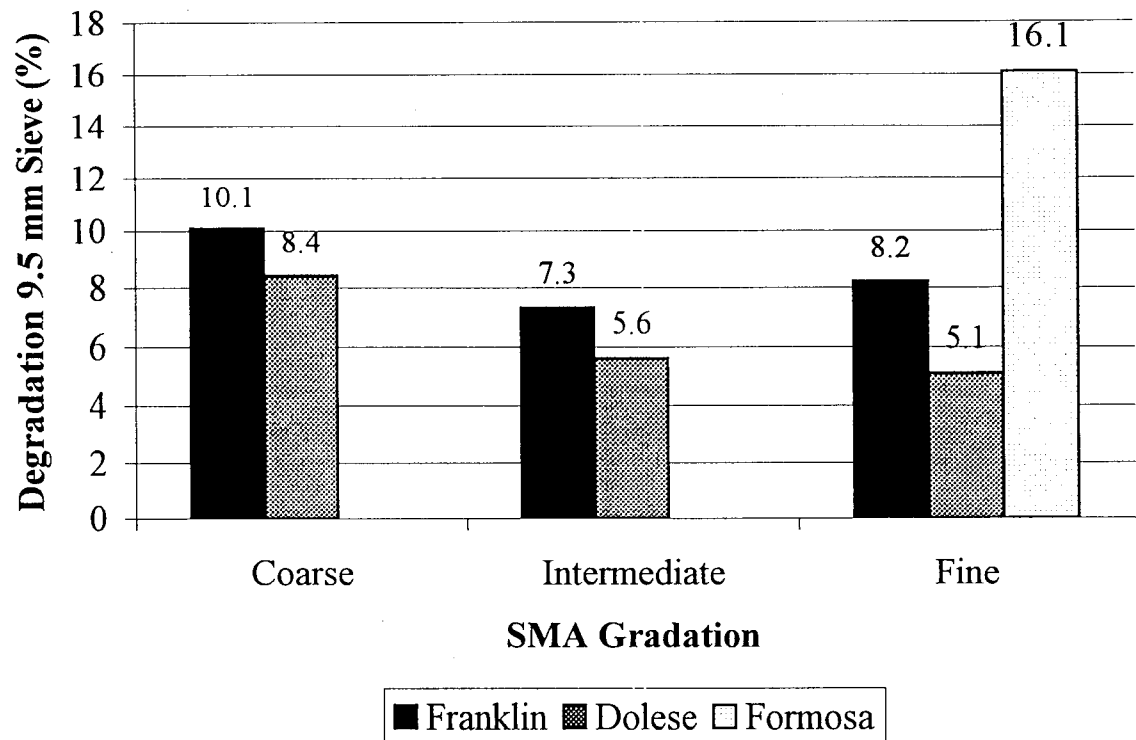


Figure 24. Aggregate Degradation by Mix.

Figure 24 shows a summary of the aggregate breakdown by aggregate source and gradation. The observed amount of aggregate breakdown was mostly a function of the aggregate source rather than gradation. The relationship between LA Abrasion and aggregate breakdown on the 9.5 mm sieve is shown in Figure 25. The relationship has an  $R^2$  of 0.60, indicating considerable scatter. The relationship shows an increase in degradation with an increase in LA Abrasion. However, to be suitable for an SMA mix, a degradation of less than 4%, would require a maximum LA Abrasion of 16%. It is doubtful if any Kansas aggregates would meet this requirement. Many states have reported good success with aggregates in SMA mixtures as high as 40%. It is obvious that LA Abrasion is a function of the aggregate source and two aggregates with the same LA Abrasion could perform completely different in aggregate degradation.

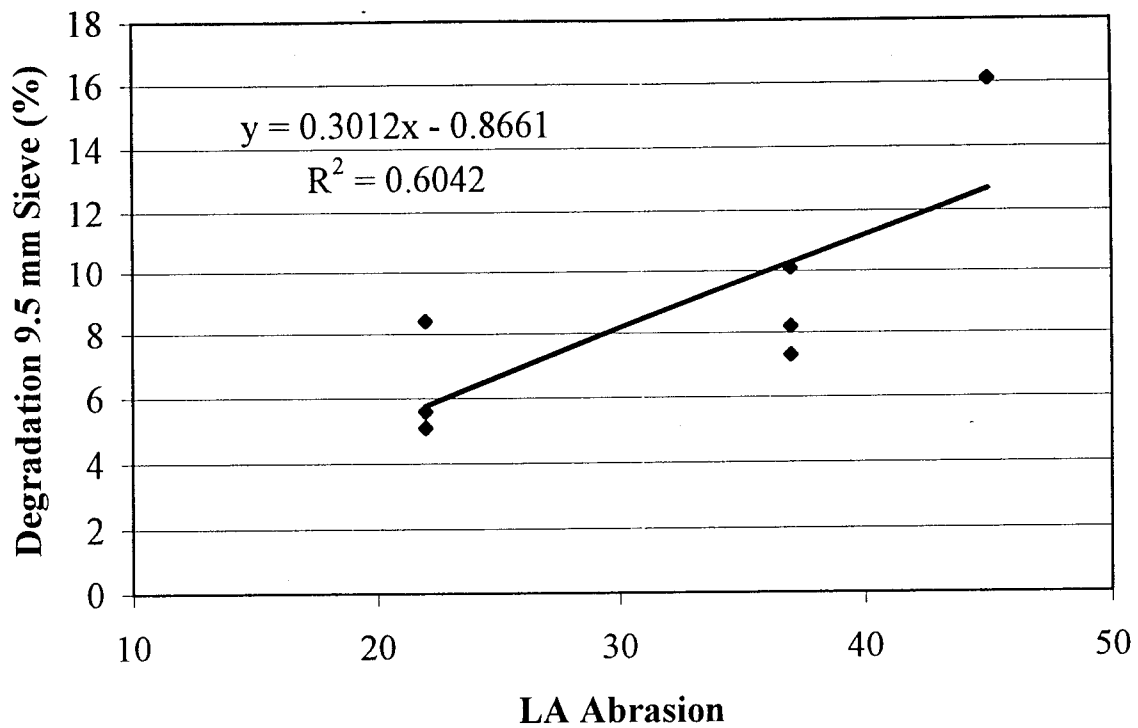


Figure 25. Degradation vs. LA Abrasion.

Although the aggregates tested did not make satisfactory SMA mixtures, the maximum requirement indicated from the above relationship is considerably below the usual maximum requirement. It is obvious that LA Abrasion is not a good indicator of aggregate performance in SMA mixtures. Burati (14) confirmed the difficulties of using LA Abrasion as a test specification with South Carolina aggregates.

**CHAPTER 6**  
**PERFORMANCE AND MOISTURE SENSITIVITY TESTING**  
**(Task 3 & 5)**

Due to the difficulties encountered in making satisfactory SMA mixtures with the aggregates selected, limited performance testing was undertaken. SMA mixtures using the fine gradation were compacted to 7% VTM at or near the optimum asphalt content using the Franklin/Riley Co. and Dolese aggregates. Primary aggregate was included.

The resistance to permanent deformation was determined using the Asphalt Pavement Analyzer (APA) in accordance with Georgia DOT Test Method GDT-115, Method A (test at dry conditions) (15). The test was performed using a 0.44 kN load on a 690 kPa pressurized hose at a test temperature of 40°C for 8,000 cycles. The resistance to moisture damage is evaluated in the APA using Georgia DOT Test Method GDT-115, Method B (test under water) (15). The samples are tested in the same manner as in the permanent deformation test, except that the samples are submerged in 40°C water.

The APA test results were compared to an SM-2C Superpave mixture made using the same aggregates and similar gradation to the Superpave mix placed on K-177 in Riley Co. (Blend 1). A second blend was prepared by adjusting the percentages of the aggregates to get a mixture with less natural sand and more VMA (Blend 2). The gradation and mix properties of the SM-2C mixtures are shown in Tables 25 and 26, respectively. The results of the APA testing are shown in Table 27.

Table 25. Composition of Dense Graded HMA(SM-2C),  
Franklin/Riley Co. Aggregates.

Material	CS-1	ManSand	SSG-1	SSG-1		
	Percent in Blend					
Blend 1	66	8	8	18		
Blend 2	70	20	5	5		
Sieve Size (mm)					Combined Gradation	
					Blend 1	Blend 2
	Percent Retained					
19.0	0	0	0	0	100	100
12.5	20	0	0	0	13	14
9.36	50	0	0	0	33	35
4.75	92	0	13	5	63	65
2.36	95	18	50	18	71	73
1.18	95	57	77	41	82	84
0.600	96	76	93	64	88	90
0.300	96	89	98	88	94	94
0.150	96	94	99	98	96	96
0.075	96	94	100	99	96.7	95.9

Table 26. Mix Design Properties, Dense Graded HMA (SM-2C).

Mix Property	Blend 1	Blend 2	SHRP Criteria*
VTM	4.0%	4.0%	4.0%
VMA	12.8%	13.6%	13 min.
VFA	68.1%	71.2%	65-78
D/AC Ratio	0.9	1.0	0.6-1.2
%Gmm@Nini	89.60%	85.80%	<89
%Gmm@Nmax	97.30%	97.80%	<98

\* SHRP Criteria 19.0 mm Nominal Mix, Traffic Level 3.

Table 27. APA Rut Test Results.

Mix	Aggregates	Gradation	Dry	Wet
			Maximum Rut Depth (mm)	
SMA	Franklin/Riley Co.	Fine	3.0	2.4
SMA	Dolese	Intermediate	2.8	2.7
SM-2C	Franklin/Riley Co.	Blend 1	7.3	N/T
SM-2C	Franklin/Riley Co.	Blend 2	3.5	N/T

N/T = Not Tested.

The results in Table 27 indicate that the SMA mixtures performed as well as or better in resistance to permanent deformation than the SM-2C mixtures with rut depths less than 3.5 mm for the two SMA mixtures and the Blend 2 SM-2C mix. The Blend 1 SM-2C mix, with 26% sand, had 7.3 mm rut depths showing the adverse effects of rounded sand in a mix when compared to Blend 2 with 10% sand. The results of the moisture damage testing indicated that neither SMA mix suffered any significant moisture damage. The SM-2C mixtures were not tested for moisture damage.

The rutting test is an empirical test and only a few state highway agencies have developed minimum rut depth requirements. These requirements range from 5 to 7 mm maximum rut depths and are for heavy traffic pavements (16). The results from the APA testing show the potential of SMA as a rut resistant mixture. However, subsequent APA testing with Kansas mixtures (17,18) indicates that APA testing at 40°C may not be adequate to differentiate performance for some Kansas mixture in permanent deformation and that testing at 50°C may sometimes be necessary to adequately differentiate mix performance.

**CHAPTER 7**  
**CONCLUSIONS AND IMPLEMENTATION**  
**(Task 6)**

An attempt was made to produce satisfactory SMA mixtures using locally available limestone aggregates. The aggregates had LA Abrasion values from a low of 22 to a high of 46. Typical specification limits are a maximum of 35% ( 8). A SMA mixture could not be made that met all of the requirements for a SMA mixture. All mixtures exceeded the maximum recommended flow value.

As a part of this research project, the limiting effect of aggregate degradation on the use of Kansas aggregates was investigated. The results indicated that Kansas aggregates degrade extensively on the 9.5 mm sieve. This aggregate degradation during compaction and/or testing resulted in excess material accumulating on the 2.36 and/or 1.18 mm sieve. The excess material was generally in excess of the current KDOT specification limit of 4 percent (2).

SMA mixtures made with lower LA Abrasion aggregates showed less degradation. The finer the SMA mixture the less degradation was experienced. The primary aggregate (chat) did not degrade. The use of the primary aggregate did not change the total amount of aggregate degradation significantly. The relationship between LA Abrasion and aggregate degradation indicated that Kansas aggregates would require a LA Abrasion of 16 or less to prevent excess aggregate degradation on the 9.5 mm sieve. This limit is unreasonable and

shows the lack of usefulness of the LA Abrasion test for controlling the suitability of aggregates for SMA mixtures.

The Dolese aggregates were placed in a SMA mix on K-254 in Wichita. The mix was placed with an asphalt content below NCAT's recommended minimum. The asphalt content of this mix was adjusted during construction as well. This leads the PI to believe that some aggregate degradation was occurring in the Dolese aggregates during production and placement. Aggregate control was performed on cold feed materials so aggregate degradation was not measured. Current QC/QA specifications require aggregate gradations on mix recovered from the road. If the same amount of degradation occurs during construction as occurred during our mix designs, then the current QC/QA specifications will be very difficult to meet.

Based on the current low to moderate traffic levels in Kansas and the performance tests of the Blend 2 dense graded SM-2C mix, extensive use of SMA mixtures in Kansas is not recommended. If KDOT desires to continue using SMA mixtures, the following implementation plan is recommended.

1. Obtain 150 mm diameter cores from existing SMA projects. Compare the extracted aggregate gradations to the cold feed gradations and the JMF to determine aggregate degradation. Evaluate if current QC/QA gradation requirements could be reasonably met using the recovered gradations.
2. Use the mix design procedure recommended by NCAT (8). After Marshall stability and flow testing, recover the aggregate and investigate aggregate degradation.

Determine if the observed aggregate degradation is enough to prevent the contractor

from meeting the QC/QA requirements. This limit could be evaluated using the information obtained in implementation step 1.

3. Evaluate the SMA mixture at optimum asphalt for performance using the APA. Consider running the dry test at 50°C to better differentiate performance. Recommended maximum rut depth, based on the available literature only, is less than 6 mm at 50°C.
4. If aggregate degradation is not excessive and the APA test results are acceptable, the mix should be suitable for use.

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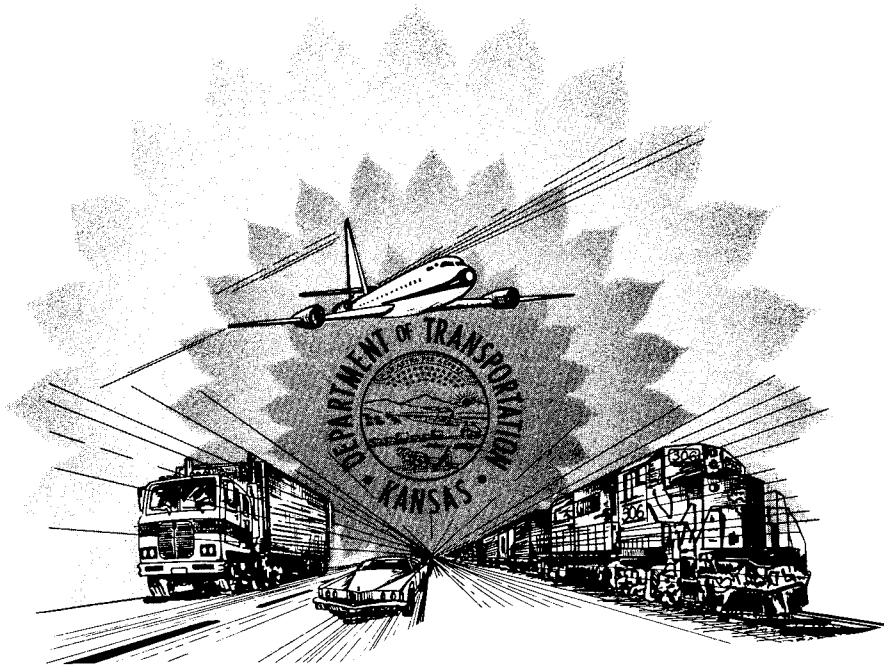
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